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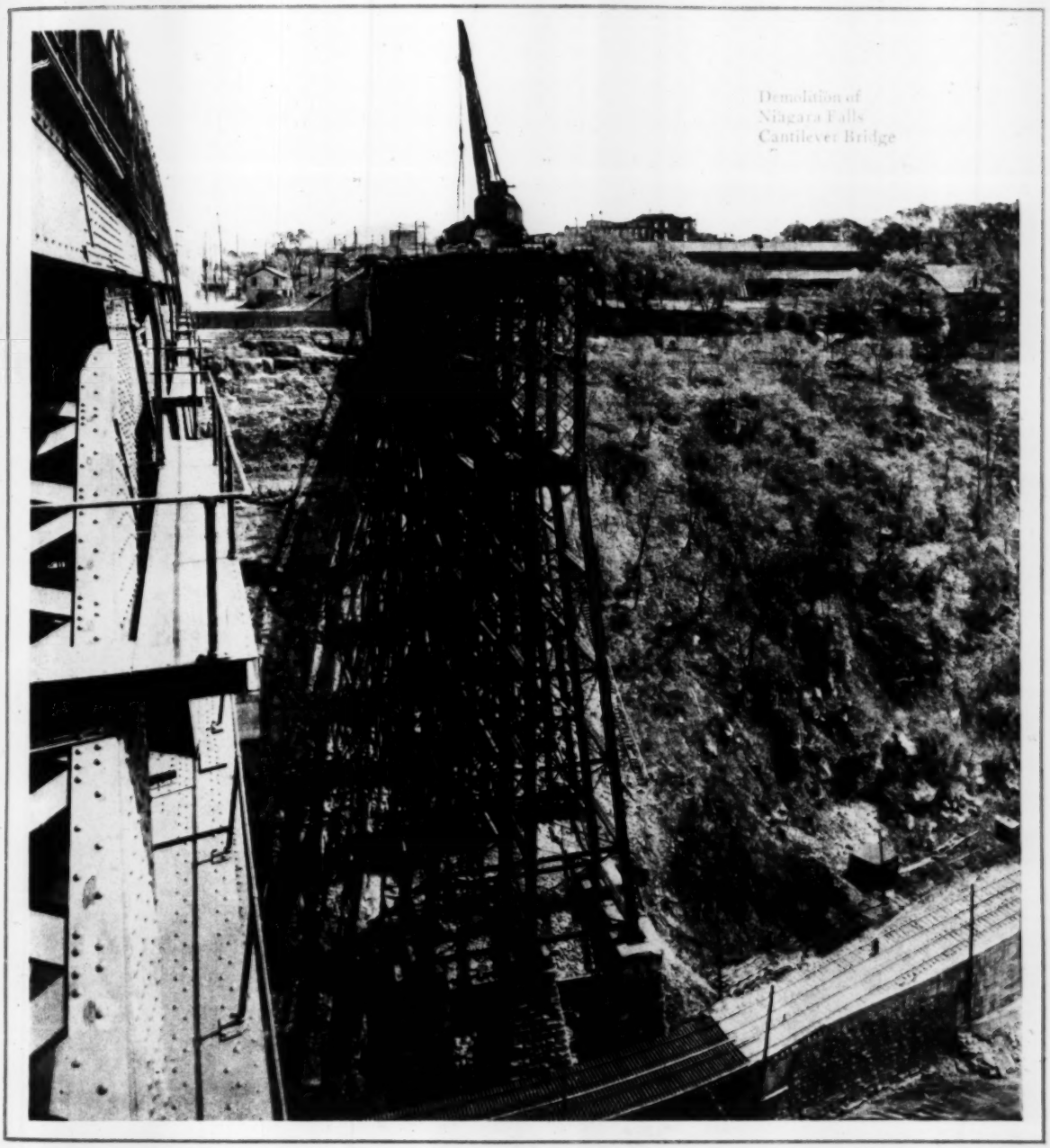

Demolition of Niagara Falls Cantilever Bridge—Concluding Article of the Series on Principles of Soil Mechanics: Development and Problem—Highway Progress in the South Atlantic and Gulf States: Mississippi—Silt and Channel Conditions in Colorado River Delta

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ENGINEERING NEWS-RECORD

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DEVOTED TO CIVIL ENGINEERING AND CONTRACTING

FRANK C. WIGHT
Editor

Volume 95 NEW YORK, DECEMBER 31, 1925 Number 27

Another Milestone

MANY men who still consider themselves young remember when *fin de siècle* was the current slang for up-to-date. To them it comes as a shock that the new century that succeeded the old whose end was thus accented has passed its quarter point. The much heralded Twentieth Century is no longer an infant; its twenty-five years are crowded with a history that surpasses the roseate predictions at its birth. Beginnings have been made in the control of matter which bid fair to change the whole life of man. We can now fly, not too securely but assuredly; we transmit words and thoughts through the air to whomsoever cares to listen; we travel cheaply and swiftly each in his own conveyance. All this these twenty-five years have seen born of man's mechanical genius. But this constructive urge has been belittled in social importance by the destructive conflict that swept across the middle sixth of this great period. In the long run radio, the airplane and the automobile will jog the sweeping curve of history much more sharply than will the Great War, but for the present their effects seem commonplace beside the aftermath of war. But wars, even the greatest of them, leave only scars on the human race. The upward trend of man is forced, in spite of them, by man's own insatiate curiosity about the laws of nature.

City Housing and Transportation

EQUILIBRIUM in city housing and transportation is an ideal regarded as possible or hopelessly impossible according to one's degree of optimism. This reflection springs from a consideration of the recent announcement of a large housing project to occupy four blocks horizontally and twenty to thirty stories vertically in the middle of the East Side of Manhattan Borough, New York City, with a frontage on a little height of land on the East River. Prominent among the merits claimed for the project is that it will help restore that equilibrium in housing and transportation so notably lacking in New York and only less so in many other large cities which results in huge reverse long-distance movements of population morning and night. The location of the new housing project is within easy walking distance of the mid-Manhattan business center development of the last decade or two, and thus—if utilized by such of the workers in that section as can pay a yearly rental of $500 a room—would lessen the north-and-south traffic congestion. The fact that the "Tudor City" project is designed to meet the needs and purses of the well-to-do should not cause those more interested in the masses of people of less rent-paying capacity to scorn "Tudor City." Those who can pay for its accommodations have a right to them. For those who cannot there are other possibilities. Between the skyscrapers of the new mid-Manhattan and the older downtown sections are "blighted districts" of large area which were passed by in the rapid march of business upward —skyward and northerly along Broadway, with recent expansion east and west from the Pennsylvania and Grand Central stations. Humbler large-scale housing projects within walking distance of Wall St. and the City Hall and sections farther north along the Broadway axis await reclamation. For the 10,000 of "Tudor City" many units of like size with lower rentals could be provided farther downtown with material relief to subway and other means of transportation. In other congested cities the same general principle, with adaptations to local conditions, could be put into practice with beneficial results.

An A.G.C. Duty

TWO meetings of importance to constructors will be held during the coming month. Illogically enough they are to be on corresponding dates and the width of a continent apart. At Portland, Ore., the Associated General Contractors will consider a program of committee reports and addresses relating largely to the business of contracting. At New York members of the American Society of Civil Engineers will organize a construction division of the society and may in some measure outline a program of activity. With the new division of construction of the American Society, a situation will be created in the organization of constructors which requires foresighted planning. It is not the purpose here to discuss possibilities or to propose a plan. The matter to be urged is that at their meeting in Portland the general contractors should arrive at some conclusion as to the logical parts the two organizations can coordinately take in their common purpose of advancing construction practice. It is not businesslike, and the Associated General Contractors is a body of business men, to allow the activities of the two organizations to drift into such adjustment as may happen. These activities must be arranged according to a constructive plan of separate and joint undertakings. The foundation of such a plan, at least in its general characteristics, is a duty of the Associated General Contratcors at their Portland convention. If it accomplishes no more than to provide a basis for discussion in working out an accepted plan, it will have been worth the effort.

More Pay for City's Engineers

ENGINEERING employees of our larger cities are proceeding in a dignified and orderly way in their campaign for a better salary schedule for government engineering work. The movement in the two largest cities has grown to include representatives of many of the employees of the next lowest in population and the extension still further down the scale may be expected. It might be thought that the campaign could be conducted by one of the existing engineering societies, but it is probably just as well to let the movement go ahead as it is started, for those who are back of it are actuated by a single purpose and can therefore concentrate their

efforts. Part of these efforts, however, should be directed toward the national societies, for they have a certain influence that the city engineering employees can not afford to neglect. Just at present the important thing is to arouse the engineering staffs of all of the cities to the necessity for courageous co-operative effort in each city. Later can come the group drive for recognition. Meanwhile all of those back of the movement may heed the warning of one of the high salaried engineers at the New York meeting two weeks ago, that it is much more important that the $1,500 to $3,000 engineer get a 25 per cent increase than that the $12,000 engineering executive be raised to $20,000.

San Francisco Bay Bridges

FOR the second time this year and the third time in four years the War Department has held a series of public hearings on San Francisco Bay bridges. This time the primary purpose was to receive evidence on the question of whether the rulings previously announced by the War Department should now be modified. Despite that clearly announced purpose, representatives of firms or individuals were present to speak for or urge approval of thirteen schemes for bay crossings, some of them brand new, all different in character and located at various points along the 6-mile waterfront between Telegraph Hill and Hunter's Point. A news report of the hearings, together with a tabulation of characteristic features of the thirteen crossings discussed, is printed on p. 1086 of this issue. The proposed Golden Gate bridge, for which a bridge district is now being organized, is in no wise involved in the present discussion, which has to do with bridges between San Francisco and the east side of the bay.

At this hearing there were even more opinions as to what War Department rulings would now be proper than were proposed crossings. The Navy and shipping interests were frankly against anything that might interfere with shipping; communities (excepting only Oakland) and civic organizations on both sides of the bay came as near to demanding a bridge as they could in a formal hearing; a bridge was shown to be urgently needed by intrastate auto traffic; several cities offered new rulings that the War Department was urged to adopt; each proponent favored rulings in accord with the requirements of the crossing proposed; J. V. Davies said that "to withhold approval of some feasible crossing on San Francisco Bay would be a stifling of the port of San Francisco in its foreign and domestic commerce," and representatives of the city of Oakland, standing alone among East Bay cities, were "unalterably opposed" to any bridge terminating on their waterfront, which they fondly expect to become "the greatest port on the Pacific Coast." The foregoing gives some idea of the jumble of conflicting demands and suggestions that are now before the War Department awaiting decision as to what will and what will not be approved.

There is a background of fundamentals, as usual, that affect all this: No location for the bridge that will not still be satisfactory twenty-five years hence is an acceptable location. The proponents of particular structures thus far have been partial, not to say prejudiced. No comprehensive, comparative study of locations and types of structures has been made. It must not be forgotten that the only direction in which San Francisco can grow and expand is southward, along the Peninsula. Finally, private capital will not and public money should not be put into any structure whose cost is so great as to impose an interest burden far beyond that justified by the traffic for many years to come.

When the War Department has reached its decision on the data now in hand, which decision can not, of course, endorse one of the specific plans but can only lay down a general basis of approval, the cities and counties which must make the final decision as to franchise for a specific bridge should get the best engineering advice available on the *comparative* advantages of the various locations and schemes under consideration. Bridging San Francisco Bay may seem a grandiose dream, but there is a persistent and growing interest in realizing the dream. The cost of such a bridge might easily exceed that of any bridge yet built in this country. The franchise granting bodies will need the best engineering advice they can obtain.

Developing an Engineering Science

IN CONCLUDING a review of his researches in soil mechanics Dr. Charles Terzaghi presents in this issue a budget of the developments that are primarily needed in order to make the science of soils a useful guide in practical engineering operations. It is a suggestive statement, adapted to stimulate thought in remarkable degree and to set in motion far-reaching investigative study.

His reference to the importance of establishing a basis for comparison of soils is particularly appealing. Our present-day art has the weakness that when the engineer undertakes to apply his observation of the behavior of one particular clay or sand soil to an apparently similar soil elsewhere he becomes involved in serious uncertainty. The second soil may be closely like the first in appearance, softness, uniformity, smoothness, plasticity or grittiness, and other properties, and yet may be quite different in its behavior under a foundation or in a trench or tunnel. Whether the two soils are of like or unlike geological origin, or have the same chemical makeup, is not necessarily significant with respect to their action under physical forces. To determine this we have to know the primary forms of physical action of soils, and the factors of makeup on which their physical behavior depends. By the help of such knowledge it will be possible not only to compare but later also to classify soils and soil types.

Early realization of this possibility depends on large-scale co-operative work by engineers and testing men. Civil engineers are already enlisted in practical soil study through several agencies. Of these, the Foundation Soils committee of the American Society of Civil Engineers is a well qualified means for bringing together and analyzing the data of soil quality. The valuable work done by this committee in its past history clears the way for undertaking this task as a new service.

The soil studies covered by the articles now concluded deal only with homogeneous soils. Such soils are of rather limited occurrence. Pure clays and sands of compact texture are the exception rather than the rule. A clay stratum, however pure and compact, does not often extend more than a few feet in thickness without lamination, parting by interposed layers, or gradual change in constitution, and even sand deposits

do not maintain uniform character for any great depth. But quite apart from such structural variation, we are more largely concerned with soils of mixed type, as loam, hardpan, sand-clay-gravel mixtures, and sands lubricated with a small content of fine material, than with single-component soils. If the mechanics of foundations and pipe trenches, of levees or of railway embankments are to become matters of precise reasoning and possibly computation it will be necessary to learn at least as much about the physical action of mixed soils as Dr. Terzaghi has developed in relation to pure soils.

Some time ago we took occasion to point out that a very broad field for investigation in soils awaits the trained research worker, a much more extensive field than one man or a few men will be able to encompass in a reasonable time. What has just been said as to mixed soils emphasizes the magnitude of the opportunity. There is room for all the workers whose interest and desire for discovery may be awakened by the evident potentialities of the new subject.

In this sense, indeed, the work so far done is only a first exploration of the field, telling of great promise and preparing for the more directly fruitful work of subsequent surveys. Such limitation may seem ungracious and unappreciative of Dr. Terzaghi's patient labors, but only to those who in using the simple but highly evolved technic of modern engineering science forget the basic contributions made by the founders of the science—such as discovery of the fundamentals of beam action or of the flow of water. The work he has accomplished in the field of soil mechanics is nothing short of epoch-making, and it can lose nothing of its brilliance and importance through recognition of the need for an even larger amount of further work.

Much Ado About an Election

EXPERIENCE with the direct primary nomination of officers of the American Society of Civil Engineers has evidently not been as productive of advantages as was thought when the revolt against the elected nominating committee brought about the present system four years ago, for three mutually exclusive amendments to the constitution each looking toward another reform are before the society. This is not to be wondered at, for the present system is clumsy and expensive, productive of elaborate and distasteful electioneering and with no apparent compensation in the way of more representative officers. That three separate groups of more than thirty members each should have felt called upon to propose a reform is sufficient evidence that the change has not been satisfactory.

The mere submission of an amendment is not sufficient reason for its adoption and when three conflicting amendments are proposed the bewildered member who is asked to vote upon them is justified in asking himself for some explanation of each one and some arguments against the proposed substitutes. It is doubtful if sufficient time will be available at the annual meeting for any such discussion, and even if there is, that meeting will be followed by a flood of printed matter in defence of each scheme. This is not a time, therefore, to discuss the relative merits of the proposals, but it is timely to suggest to every member of the society that he should read them carefully and weigh not only their evident intent but also their implications.

This member will find that in two of the proposals the nomination of the president and vice-presidents only is changed; directors continue to be both nominated and elected by direct ballot of members from their district, vice-presidents elected by direct ballot of members of their zone and the president elected by direct ballot of all members. In the other scheme, placed first in order in the official notice, there is set up a nominating committee of five past-presidents and representatives from each district, which committee makes all nominations, of directors as well as executive officers, the elections to follow by direct ballot as at present.

The two other methods are radically different. In one the board of direction is ex officio the nominating committee, for executive officers, and in the other there is set up what might be called a second house, of representatives of the local sections, which has as a part of its duties the nomination of the executives.

The member will note, too, that the first amendment substitutes for the four successive ballots now required two ballots and one notice and a general meeting of twenty to thirty men which the society pays for; that the second alternative has no extra meeting to pay for but still requires four successive ballots and that the last alternative requires three ballots and a general meeting of as many men as there are local sections, which may well cost the society three or four times what one meeting of district representatives would cost. So far as cost and complication goes, then, none of the methods improves over the present system and one, at least, will be much more expensive.

Whether any of the methods will discourage electioneering or provide more, or less, representative officers—the competitive issues in the supplantal of the old nominating committee by the direct primary—the inquiring member will doubtless begin to form his own opinion and until the competing arguments are presented comment to that end may well be withheld. One cannot avoid seeing, however, that the last proposal is much more than a revision of nominating methods; it is a radical revision of the government and possibly the purpose of the society. On that account it should be examined carefully and discussed fully.

Under this amendment membership in a local section is made compulsory, regardless of actual residence or desire for affiliation. The representative of the section is vested with considerable powers, by constitution in the nomination of officers and by implication in the general conduct of the society. A complicated form of government is set up, with one elected group governing the society and another group, elected by the same members differently distributed, nominating an influential section of the governing group and exercising vague, but no less real, powers of control. The member considering the amendments may well inquire why this involved machinery of government for a professional society.

At any rate it is obvious that a considerable number of members of the society want a change in the methods of electing officers and that something interesting will develop at the annual meeting and in the ensuing months of circularizing. It behooves every member of the society to study the alternatives and make up his mind which is the best, or whether the existing method, unsatisfactory as it is, has been successfully reformed.

Demolition of Niagara Falls Cantilever Bridge

Expeditious Fieldwork in an Unusual Undertaking Complicated by Difficult Location—Three-Truss Bridge Taken Down by Reversal of Cantilever Erection Method

By H. Ibsen
Bridge Engineer, Michigan Central R.R. Co., Detroit

ON FEB. 16, 1925, the new 640-ft. steel arch bridge carrying the Michigan Central R.R. across the Niagara Gorge at Niagara Falls was opened to traffic. Two months before, a contract for the demolition of the old cantilever bridge, alongside, had been let to the American Bridge Co., on a cost-plus-sliding-profit basis. This demolition work, completed on Sept. 1, will be described as an interesting instance of dismantling by reversal of the cantilever erection procedure.

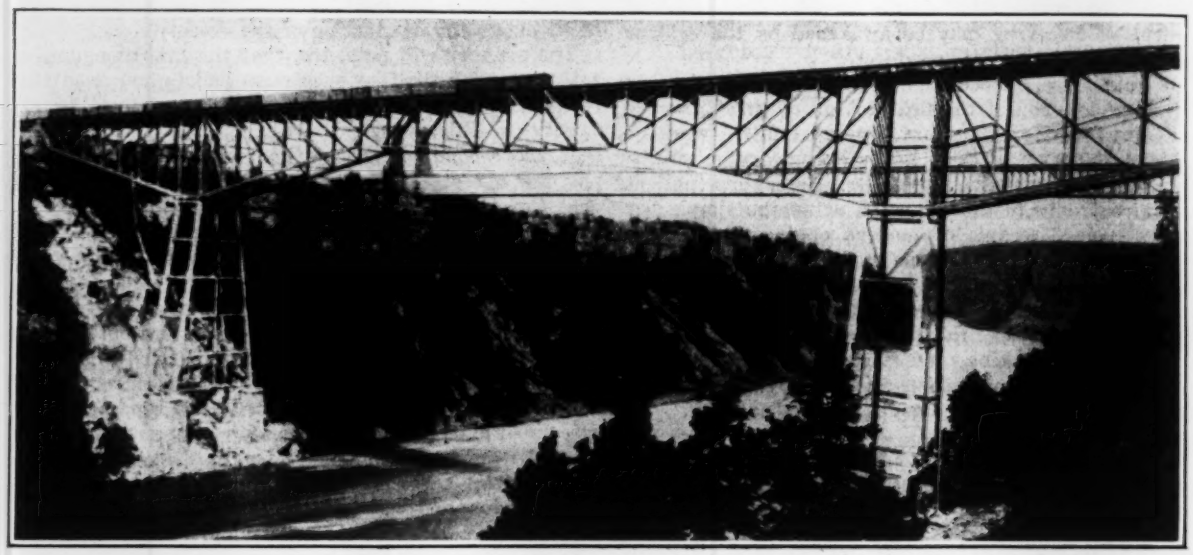


FIG. 1—NIAGARA CANTILEVER BRIDGE AS ORIGINALLY BUILT
Old suspension bridge in background replaced in 1897 by 550-ft. double-deck steel arch built for both railway and highway traffic. Cantilever replaced by 640-ft. steel arch opened to traffic Feb. 16, 1925

Both the cantilever and the new arch were described in *Engineering News-Record* last year (March 1, 1923, p. 380, and March 6, 1924, p. 398). A view of the cantilever soon after its construction in 1883 is given in Fig. 1, from an old photograph, and an outline elevation is shown in Fig. 2.

Choice of Method—Two methods of demolition were considered:

First, Reversing the method used in erecting the bridge, that is, erecting falsework under the shore arm and starting at the center of the suspended span, taking out the bridge piece by piece with a self-propelling crane, loading the material on cars as taken out and backing toward the shore;

Second, Removing the bridge without falsework under the shore arm, which would mean balancing the two arms on the towers, as first one panel in the river arm was removed and next one panel in the shore arm was removed.

The second method would have meant building a special traveler with booms at either end and lowering the material down on the steep river bank, snaking it up the bank and hoisting it from the foot of the bluff to track level for loading.

While falsework would be costly, considering the expense in preparing good foundations, the cost of the special traveler and the extra handling of the material would probably balance most of this, and in addition the second method involved greater risk of accident. For these reasons it was decided to adopt the first method.

Falsework—The wooden falsework (Fig. 2) was framed in the storage yard. Each 30-ft. bay was spliced on the center line of the bent, so that each half bent section could be shipped connected up from the yard to the bridge on flat cars and there picked up by the crane and lowered to its position in the bent. Heavy struts following the slope of the bank were wedged between the feet of successive bents, butting against the tower piers, to prevent any downhill movement of the bents. The foundations had to be dug much deeper than shown on the drawing, and on this account the bottom bays of the bents had to be much longer and they were set on a continuous bed of mud sills. Work was started on the bent next to the tower and the mud sills and bottom bay of the bent were placed before starting excavating for the next bent up the hill. The excavated material from the next footing was then wasted down hill, so that it buried the footing of the bent below by 8 to 12 ft., dependent on the soil conditions found. All together about 5,000 cu.yd. of material had to be moved, much of it being boulders, some so large that they had to be drilled and broken up before moving.

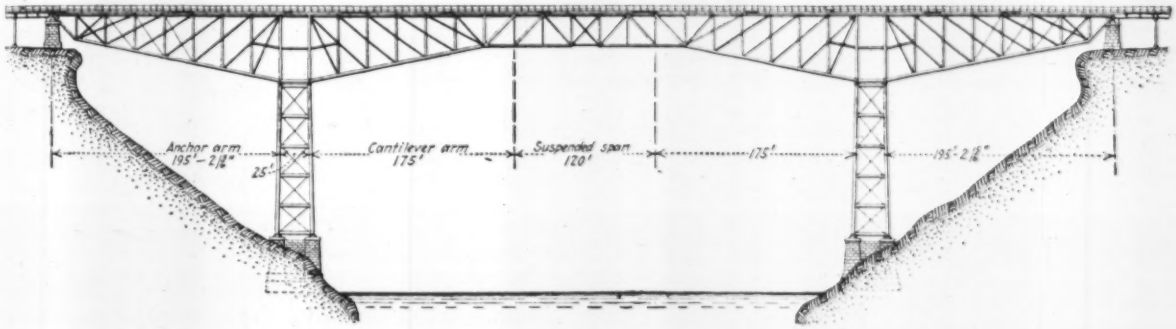
As a matter of economy the falsework used on the American side was used over again for the Canadian side. This increased the time for the work about three

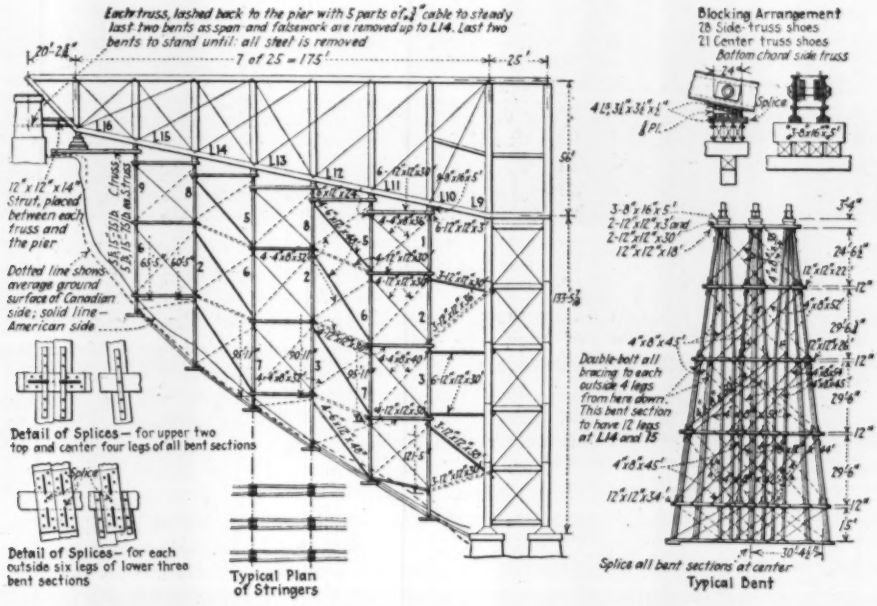
weeks but it saved about 200,000 ft. b.m. of timber and the labor of framing. Falsework construction was started at the end of February, 1925, and carried on as the weather permitted. Removal of the superstructure began in March.

Preparing the Bridge—The first thing done was to rearrange the bridge floor so as to have one track on the center line of the bridge. In 1901 an extra line of stringers had been added under each track between the two old stringers; the outside line of old stringers were therefore removed, and the two lines of 1901 stringers were moved in, close to the inside lines of old stringers, and ties were placed close together on these four stringers, which gave a good support for the heavy concentrated loads of the demolition cranes. The members of all three trusses could be easily reached and handled from this center track and, as the crane could swing clear around, the car to receive the members removed could be placed back of the crane. Except for some of the 1901 stringers, the material could be used only for remelting; therefore the members were all burned apart to suitable dimensions for shipment, and the material was sold as loaded, so that no rehandling was necessary.

By the end of April the rearrangement of the floor was completed and the falsework (under the American anchor arm) had been finished. To bring the trestle bents to bearing, the pressure from a 35-ton jack was applied under each panel point of the trusses, which closed up the joints between the timbers of the bents and brought the footings to a good bearing. There was very little settlement after the falsework carried the full load of the shore arm.

For removing the bridge members two self-propelling cranes weighing 195,000 lb. distributed on four axles were used. The heaviest member to be removed weighed 26,000 lb., and this the cranes could safely handle.




Each truss, lashed back to the pier with 5 parts of 2¼" cable to steady
last two bents as span and falsework are removed up to L14. Last two
bents to stand until all steel is removed
7 of 25 = 175'
25'
L16
L15
L14
L13
L12
L11
L10
L9
12"x12"x14' Strut, placed between each truss and the pier
Dotted line shows average ground surface of Canadian side; solid line—American side
Detail of Splices—for upper two top and center four legs of all bent sections
Splice
Detail of Splices—for each outside six legs of lower three bent sections
Typical Plan of Stringers

Blocking Arrangement
28 Side-truss shoes
21 Center-truss shoes
Bottom chord side truss
Splice
Double-bolt all bracing to each outside 4 legs from here down. This bent section to have 12 legs at L14 and 15
Splice all bent sections at center
Typical Bent

FIG. 2—MICHIGAN CENTRAL CANTILEVER BRIDGE, AND DEMOLITION FALSEWORK

It was desirable to keep the work on the Canadian side going while waiting for the falsework. But in the bridge as built this met with the difficulty of stress reversal in the top chord of the anchor arm as soon as the suspended span had been removed, at which time the compressive stress set up by the crane with a loaded car moving over the shore arm would overcome the dead-load tensile stress due to the weight of the river arm. The old (outer) trusses had a top-chord section consisting partly of eyebars and partly of a built-up member, which latter was stiff enough to take care of the compressive stress, but in the center truss (a later addition) the first three panels back of the tower were made up entirely of eyebars and therefore could not be relied upon to take the compression. To correct this condition closefitting wooden blocks were driven between the eyebars at intervals of 2 ft., and 1-in. rods were placed above and below the eyebars at the blocking to tie them together. The three panels were thus made safe for compressive stress, so that the entire river arm could be safely removed and the loaded cars switched over the shore arm by the crane.

Taking Down Suspended Span—While this work was being done the floor and stringers had been removed from the suspended span. The outside trusses of the suspended span were connected to the ends of the cantilever arms by struts with slotted pinholes in both top

and bottom chord, but the center truss had such a strut only at the bottom chord and no tie in the top chord. The slotted pinholes in the struts were tightly wedged with shims cut to fit the pinholes, and the hip joint of the center-truss top chord was tied to the end top-chord pin of the cantilever with six lines of ¾-in. wire cable. To stiffen the center truss while the side trusses were being removed, timber struts were placed between the top-chord hip joint of the center truss suspended span and the end joints of the outside trusses of the cantilever arms.

The top-chord lateral bars of the suspended span were now removed; the diagonal truss bars in the center panel (T2B2–T2′B2′) of the outside trusses, and the rod swaybracing at T2B2 and T2′B2′, were slacked off, and the bottom laterals in the center panel were cut and dropped into the river. The two cranes, standing at the end of the cantilever river arms, then took hold of the top chord of one of the side trusses of the suspended span and the chord was burned off close to the hip joints (U1 and U1′). The cutting of the chord was done in such a way that with the rods slacked off, as mentioned above, the truss would gradually deflect to about the position it would assume with the top chord removed. It was necessary to burn through the chord in the same place several times before the cuts remained open enough to permit the chord to be removed. The vertical posts T2B2 and T2′B2′ were then cut between underside of chord and top of floor-beam (the floorbeams were riveted to the posts about 6 in. under the chord), and the chord was lifted and swung over close to the center truss where it was laid down, burned through at its middle, and the halves were removed to the scrap cars back of the cranes. Then the center panel diagonal bars T1B2′ and T1′B2 were cut and dropped into the river as well as four of the six bottom-chord eyebars in the center panel.

The same process was repeated for the other outside truss. Now temporary wooden struts were placed on top of the floorbeams T2B2 and T2B2′ in panels T1T2 and T2′T1′, to hold bents in place when the center-truss top chord was removed. The top chord of the center truss was cut and removed in the same way as the outside trusses.

Posts T2B2 and T2′B2′ in both outside trusses were taken out and loaded, and thereupon the floorbeams at T2 and the same posts of the center truss were removed. Then the bottom laterals in the panels next to the center panel and the remaining two bottom-chord bars in the center panel were burned off and dropped into the river. The last chord bars were burned through gradually, with the idea that the bars should stretch when the remaining section became small enough, if there still was any appreciable amount of pull in them, so that they would not break suddenly producing impact in the struts supporting the remaining parts of the trusses. The precautions taken were effective and there was very little movement when these members were finally cut through and the center panel of the suspended spans was completely removed. (In all the work over the river the lighter members, such as bars and small struts, were dropped into the river when their value as scrap was not enough to pay for the labor of handling and loading them.)

Two days' more work removed all of the suspended span except the hanger rods and bottom struts at the ends of the cantilever arms, and the cranes were moved back to U2 ready to remove the first panel of the river arm (Fig. 3).

Rapid Work on River Arms—Removal of the river arms was commenced May 6, and was completed by May 26, when the tower panel on the American side was taken in hand. On the Canadian side two panels remained, one of which was left in place until the falsework was set under the anchor arm.

There was nothing to deserve detailed description in the work of removing the river arms and the towers. It became a work of routine. The burners would keep as far ahead as practicable, burning part way through a member; then the main falls would be hitched to it (the secondary fall supporting such other pieces as required temporary support), and the member would be severed completely, hoisted and loaded on the car placed behind the crane. This procedure was repeated until the panel was all taken out and the crane moved back another panel.

The riverside truss bent U8L8 and the tower bent below, with all the tower bracing, on the American side, were taken down next. The work began May 26 and the castings at foot of tower were hoisted away June 5.

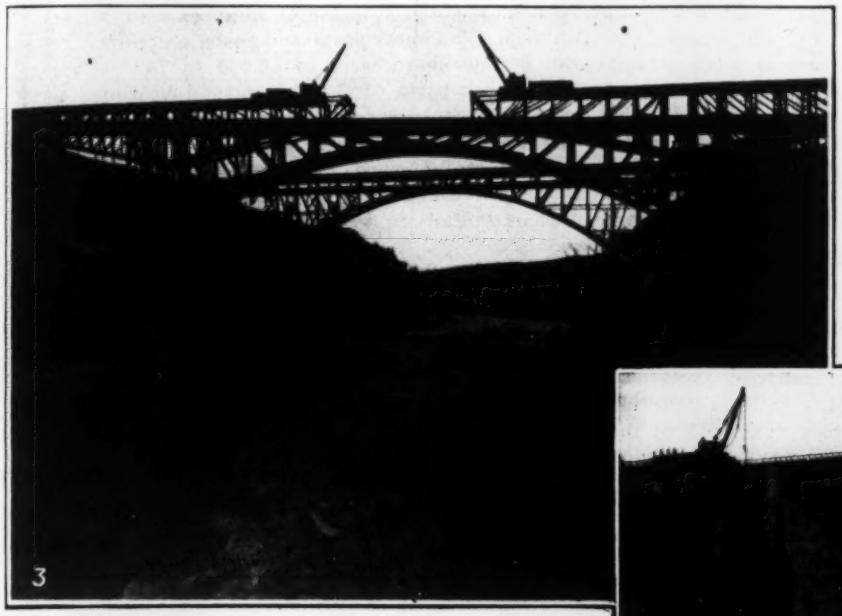
Rusting of Pins and Bars—The posts of the truss bent U8B8 had a half pinhole at the bottom, where they bore on the pin in the casting on top of the river-side tower leg. Upon lifting the outside posts of this bent from their bearings we found that, for erection purposes, a 1-in. square piece of iron had been placed under each corner of these posts. These little blocks I think originally were about ¾ in. thick. Presumably the erectors expected the blocks to squeeze out flat under the load and permit the pin to come to a bearing, but this had not happened. The blocks were squeezed down somewhat, and at some points they were forced between the several plates of the posts, spreading the plates apart. Where they had squeezed down enough to allow water to get at them, the blocks had become flakes of hard rust, and the post had settled down to a bearing on a small part of the pin; in all other parts the post, including its center diaphragm, had no bearing on the pin except for such layers of hard rust flakes as had formed between pins and pinholes of the post.

These points were inaccessible to examination and I always expected to find these as well as other similar places on the bridge badly rusted, which proved to be the case. After hammering off the scale both pins and plates of the members were found to be pitted as much as ¾ in. deep in large spots. The same condition existed on the Canadian side.

Except for the outside top chords of the river arm, where the chord was a combination of a built-up member and eyebars and where for five panels the reversals of stress were large and there had been considerable wear in the pinholes, the eyebars in the chords and diagonals were rusted so firmly to the pins and to the adjoining bars—especially where, as in nearly all cases, they were packed so tight as to be inaccessible for cleaning and painting—that when one end of the bar was burned off the weight of the bar would not cause any movement on the pin at the other end. These bars were 8x1¾ in. in section and 22 ft. long from center of pin to the burned end. In one case I tried a joint, with eyebar stubs sticking out 3 ft. from the pin, by first striking the bars numerous blows with a heavy maul to loosen the rust and then hitting the end of the bar. Treated in this manner it moved only ¾

in. in six blows of the maul. In driving some of the bars off the pin they were found encrusted with scale to a depth of nearly ½ in. in spots, so that it would have been a very slow and expensive job to drive out the pins, which were 6 ft. long and carried 32 eyebars.

Work in Anchor Arm—On June 8 the crane was moved back off the river arm and the diagonal bars U9L10 from top of shore tower bent to the bottom of the next panel were cut, transferring most of the weight to the falsework. The crane was then moved back to panel point U10 and took out the top chord in panel U9L10 and the diagonal bars U9M10, which brought all the load on the falsework. Levels and alignment of trusses and falsework, observed regularly as the work progressed, showed a small settlement in some of the bents, but too little to require any correction to be made.



FIGS. 3 AND 4—DEMOLITION WORK: SUSPENDED SPAN REMOVED (MAY 25)—LAST TWO PANELS BEING TAKEN DOWN (AUG. 27, 1925)

The shore bent of the tower and the first truss panel were down by June 16. From this point the work went slower, as the falsework had to be taken out as soon as the panel above was removed, before the crane could move back.

By July 13 the shore arm on the American side and all of the falsework that had to be used over again on the Canadian side had been completely removed and on July 25 the falsework on the Canadian side had been completed and work started on removing the shore arm, tower and the one panel remaining of the river arm of the cantilever on that side of the river. The Canadian anchor bars, which were the last part of the main bridge, were removed on Aug. 29, and two days later the falsework had been removed and the work was completed except for the removal of the street bridges and for the cleaning up of the bridge site.

Some Service Experiences of the Bridge—One source of trouble with the old cantilever bridge was the floorbeams (see a reference to this trouble in *Engineering News-Record*, March 1, 1923, pp. 380-381). Where these were riveted to the posts of the outside trusses, the end angles connected direct to the floorbeam webs and did not, as in modern construction, extend over the legs of the flange angles of the floorbeam. In addition to this the top lateral rods were attached to the floorbeams through a pin, the center line of which was 20 in. below the center of the chord and 21 in. from the center of the truss. The pin was connected to the floorbeam by a plate fastened to the top flange of the floorbeam and by a bracket riveted to the web of the beam below the flange, and thus there was no means of transferring the stress from the laterals to the chord except through the floorbeam. In 1907, to take care of the horizontal component of this stress, a lateral chord was attached to the top of the floorbeam for four panels each side of the tower. While this helped the conditions for which it was put in, it rather aggravated other defects.

The floorbeams were riveted direct to the posts of the outside trusses as described above, and when the new center truss was added (in 1900) the new posts were put in without cutting the floorbeams, except in the tower bents. Shims were placed between the floorbeams and a seat on the new center-truss post to transfer to the center truss its computed share of the dead-load. Then the floorbeams were riveted to the center trusses, making them continuous over the center truss and thereby imposing upon them the additional duty of helping to maintain equal profiles of the three trusses under live-load. In addition to this there were four lines of stringers under each track, resting on top of the floorbeams. These were supposed to slide freely on their bearing plates, but in spite of raising the ends from time to time and greasing the bearing plates, their movement was not entirely free. As the stringers and the added lateral chord did not take part in the movement of the trusses under live-load, they restrained the floorbeams and produced transverse bending stress in them.

These causes combined had the effect of cracking

the webs of the floorbeams, the break generally starting just above the end connection angles. In 1911 the breaks had become so numerous, and were so rapidly getting worse, that repairs had to be made. This was done by first replacing the brackets, under the floorbeams, which formed the swaybrace connection to the floorbeam and the truss post; the new brackets were stronger and deeper than the old ones and had enough connection rivets through the post to support the floorbeam load. Then the end angles and the bracket connecting the top laterals to the floorbeam were taken off, and heavy plates, running the full depth between the horizontal flanges of the floorbeam angles, were put on; the ends of these plates were bent to form angles fitting against the face of the post, and were connected by the old floorbeam connection holes in the posts. These plates, 2½ ft. wide over the top flange angles, formed an ample splice between the split web and the angles. Where these plates connected to the truss posts, the upper holes were fitted with shoulder bolts, the holes in the plates being ⅞ in. large, to provide for movement in the upper part of the floorbeam, while the lower half of the connection was made with tight bolts.

When these floorbeams were removed there was further evidence of the movement that had previously given trouble, in the form of splits in the connection plates at the bolt holes where they connected to the post. In some places there was a double split, allowing a piece of the plate opposite the bolt hole to drop out.

Some time ago we found that one of the steel castings supporting the center truss on the anchorage pier had split, under the center line of the pin to which the anchor rods were attached. The two halves of the casting had moved apart at the bottom about ½ in., and directly under the pin about ⅞ in. As it would be very difficult and expensive to renew the casting, as the anchor bars ran through it, it was repaired by drilling holes through the webs and putting in four 1½-in. bolts tying the two halves of the casting together. Frequent inspection was given the casting but there was no further movement apart of the broken halves.

The cause of the break was always a matter of speculation, as there was no sign of defect on the surface. When the casting was removed we found the trouble. The webs, especially the center web, which was 6 in. wide, contained large segregations of very coarse crystals. In the center web a piece 6x3x½ in. dropped out when the two halves came apart in removing the casting.

Corrosion Experiences—It was evident for some time that the material of the center truss (added in 1900) suffered much more from rust than the old (1883) trusses. This was particularly noticeable on the lacing of the tower struts. As early as 1911 there were holes rusted through the lacing bars of the longitudinal struts of the central tower legs. The lacing bars of the members in the outer lines were somewhat pitted, but were not in very bad condition. By 1918 the lacing bars of the middle tower struts had become so bad that several hundred of them had to be renewed, while only three or four of the bars of the outer struts were renewed and even these were not in as bad condition as the former. I think that the lacing bars of the outer struts were iron, and of the center struts medium steel.

The center truss was in somewhat worse position for catching brine drippings from refrigerator cars. The outer trusses also suffered considerably from this action, but relative to their length of service they showed much better resistance to deterioration than the center truss.

The outer or old trusses were built partly of high-carbon steel (average tensile strength 83,000 lb. and elastic limit 54,000 lb. per sq.in.), and partly of wrought iron. The high-carbon steel was used for tower posts, end posts, posts over towers, and bottom chords of cantilever arms; other material was wrought iron. The later middle truss was also built of two grades of material. The tower posts, end posts, posts over towers, and bottom chord were of 66,000 to 74,000-lb. steel, while other parts of the trusses and bracing (except lateral and sway rods, which were iron) were made of 57,000 to 66,000-lb. steel.

Paint maintenance had at all times been quite thorough. Since 1900, at least, the bridge received as much attention in the way of painting as would be required by the most stringent maintenance specifications. Except within the last couple of years, since work began on the new bridge, the cantilever bridge had been patched up almost yearly, and repainted completely at short intervals. The main parts of the material were in very good condition. The rapid destruction of the lacing of the tower struts was probably due largely to the force with which the brine drops would strike them, because of the height of fall. In respect to this action, the struts of the old and new tower bents were, of course, on equal footing.

Other Notes on Service—Service experience with this bridge leads the writer to advise against the combination of compression and tension members as used in the top chord of the shore arms of the old trusses. The compression and tension elements of these members did not work together, and it is probably not practicable to make them work together in any case. Further, in eyebar tension members having many bars packed on a pin, arrangement should be made to separate the bars either far enough for cleaning and painting or else far enough for oil to penetrate between them. A housing should be put over the bars at the pin joints to keep dirt from accumulating between the bars, so arranged that oil-soaked waste could be put within the housing over the bars.

Where floorbeams are riveted to the vertical posts of the trusses, it is desirable to have positive means to assure the free movement of the floorbeams with the trusses under load. This could perhaps be done most easily by putting rollers under one end of the stringers in each panel. The connections of the lateral bracing should be such that the laterals would not interfere in any way with the free movement of the floorbeams, and at the same time to transfer the lateral stresses directly to the chords.

Hip joints and end joints of anchor arms in a cantilever such as this one should be riveted joints, as there is considerable motion and resulting wear at these points.

Approach Spans—While the work of removing the cantilever span was going on, preparations for removing the two street spans in the approach on the American side and one on the Canadian side were made. One of the American bridges was a double-track half-through-girder span 66 ft. long, with two main girders into which were riveted deep transverse beams carry-

ing a 9-in. concrete slab. The other two bridges were short spans with I-beam and plate ballasted floors. A very good quality of concrete was found in the slab, and removing it proved difficult, especially as the span was over a public street with car track. Extreme care was taken and the concrete was drilled and cut out in small sections. The contractor's foreman rigged up a very efficient tool for drilling and cutting the concrete, by making special drills and chisels to fit the large pneumatic rivet-busters, which saved considerable time.

The steel on which the concrete rested was found in absolutely perfect condition, with its red-lead coating still showing its bright color. The reinforcing steel in the slab was as good as new, showing no rust. These bridges were all built in 1911, to replace the original iron trestle approaches which had never been strengthened and which were in very bad condition at that time.

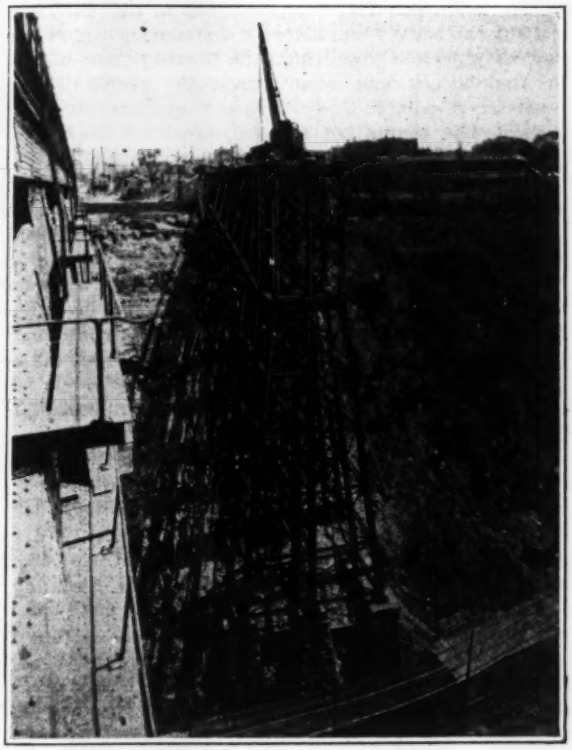


FIG. 5—AMERICAN TOWER AND ANCHOR ARM; RIVER SPAN REMOVED (MAY 26, 1925)

The waterproofing on these bridges, which consisted of three layers of raw burlap laid in "Sarco" asphalt and covered with 1½-in. asphalt mastic protection coating, was in remarkably good condition, after fourteen years' service under heavy traffic. The asphalt apparently was as good as when it was put in, and the raw burlap, which is generally said to deteriorate very fast, was as sound as new, as far as I could judge.

The work of removing the cantilever bridge was successfully completed without accidents of any kind and with only few minor injuries to the men; this was due to a large extent to the care and foresight of the American Bridge Co.'s foreman, S. F. Frantz, who had charge of the work. The men who handled the torches for burning off the steel suffered a good deal from the fumes caused by the burning of the old paint of the members, and a couple of them had to go to the hospital to recuperate.

At the present time the anchor pier on the Canadian side and the old concrete abutment for the Front St. crossing in Canada are being removed by the Federal Construction Co., Toronto, Ont.

James H. Curtin has had charge of the field work under my supervision.

Model Tests of Wind Pressure on High Buildings

AS A result of many inquiries about wind pressure on buildings the U. S. Bureau of Standards recently undertook some experiments to determine the amount and distribution of such pressure. There appeared to be a general feeling among engineers that the values of wind pressure commonly used in the design of engineering structures are too high. The experiments on which these values are based were carried out many years ago, by relatively inaccurate methods, and on models not resembling actual structures. The success of modern wind tunnel methods in the field of aeronautics suggested their use for the determination of wind pressures on models of structures. Experiments were therefore made on a model in the 10-ft. wind tunnel of the Bureau.

The first model investigated was a model of a tall building, in the form of an 8 x 8-in. square prism 24 in. high. The pressure due to winds of speeds up to 70 m.p.h. was measured at 70 stations on one face and 49 stations on the top, for wind directions varying from 90 deg. to one face to 90 deg. to the opposite face by 15-deg. steps. The effect of the ground was represented in two ways, first, by the floor of the tunnel, and second, by a fixed platform. The pressures were integrated to give total forces and moments on the separate faces and on the complete model.

The chief conclusions of interest to structural engineers are as follows:

(1) The greatest average pressure against the building occurs when the wind blows normal to a face and is equal to 1.5 times the velocity pressure, i.e., 22 lb. per sq.ft. for a true speed of 76 miles per hour (100 miles per hour indicated by a Robinson type anemometer).

(2) The average decrease in pressure over the roof is about 0.83 times the velocity pressure, i.e., 12.2 lb. per sq.ft. for a true speed of 76 miles per hour.

(3) The maximum moment about a vertical axis is equivalent to a horizontal displacement of the resultant force of 0.077 times the width of the building.

While the above conclusions apply exactly only to geometrically similar structures, they apply approximately to structures of the same general shape.

A full report on the tests will be published shortly as a paper in the scientific series of the Bureau. It will include a general survey of the effects of steady wind and some discussion of allowances for gustiness.

Known Sources of Typhoid Only 60 Per Cent

It is a wise and fortunate epidemiologist who can determine satisfactorily the source of infection of more than 60 per cent of the cases of typhoid fever occurring in a given city, declared Dr. Louis I. Harris, director Bureau of Preventable Diseases, New York City, in an address on the control of communicable diseases before the New Jersey Sanitary Association on Nov. 30.

Principles of Soil Mechanics: VIII—Future Development and Problems

Origin and History of Experimental Study of Soils—Future Objectives—A Theory of Models—Soil Classification—The Method of Equivalents for Engineering Practice

By Dr. Charles Terzaghi
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Temporary Lecturer, Massachusetts Institute of Technology, Cambridge

In contrast to the rapid advance of engineering science in the fields of statics, dynamics and hydraulics within the last century there has been complete stagnation in the study of soil mechanics. Since Coulomb and Rankine formulated their classical theories of granular earth pressure practically no progress has been made, and the vast field of the mechanics of plastic soils has hardly been touched. Earthwork engineering is in consequence about on that level of efficiency in design at which mechanical and structural engineering found themselves a couple of centuries ago. The reason for this lack of progress lies mainly in the great difficulty of investigation of soil phenomena. In recent years, however, various investigators have turned their attention to the subject. A summary of experimental results obtained by the author has been presented in the preceding articles of this series. Further activities along similar lines are planned, in several institutions of this country, and it may be said (speaking somewhat optimistically) that a general attack upon the unsolved problems of soil mechanics has begun. It will therefore be of value to review the situation briefly.

Engineering Geology—The first efforts were made in the direction of developing engineering geology. A text-book on the subject was written as early as half a century ago (by Dr. D. Braun), though from the present standpoint the work was of little value. Within the last generation or two, many engineers have recognized the need for a better bond between geology and engineering, and have worked to bring it about. Italy and Belgium created special departments for engineering geology. Other states sent their geologists to every important construction enterprise, in order to bring geology and engineering into constructive contact. Meritorious works on engineering geology have been written. Yet the results of the movement are on the whole disappointing.

The more one penetrates the field of engineering geology, the more its innate limitations become evident. In its present state, the science is of service in such matters as locating the sites of structures, or selecting the number and location of test borings, but this is after all only a small part of what is needed. We obtain no exact information concerning the strength and physical properties of the materials composing the several geological strata, and we do not yet find in the teachings of geology any suitable basis for identifying and classifying soils.

Experimental Soil Studies—It thus came to be realized in the course of time that soils must be studied physically as well as geologically, if we are to have a substantial basis for earthwork engineering. It is to the credit of the American Society of Civil Engineers to have made the first conscious step in this direction. In 1913 the society appointed a foundation committee "to codify present practice on the bearing value of soils for foundations and report upon the physical characteristics of soils in their relation to engineering structures." The committee evidently realized the importance of its task; it has displayed very fruitful activity throughout its existence. By an interesting coincidence, in the same year the Swedish Government Railways appointed a Geotechnical Commission for the purpose of investigating the physical characteristics of materials in unstable slopes and devising remedial measures for slides. Thus, the need for soil investigation found simultaneous recognition on two continents.

During its ten years of existence, the Foundation committee has made many interesting reports. It has conducted friction tests, investigated the compressibility of various soils, and measured the compressive, tensile and shearing strength of undisturbed clay samples, among other things. No general conclusions have yet been reached, and in view of the lack of knowledge concerning the basic relations between the properties of soils it is impossible to make much practical use of the data obtained. The reason is obvious: Each one of the so-called physical properties of soil—internal friction, cohesion, plasticity, compressive strength, and the like—is in itself so utterly complex a phenomenon that it cannot be correlated with any of the others. It would be necessary first to discover the elementary physical factors which determine these properties. Lacking a knowledge of the elementary factors, experiments merely build up a collection of unusable data and give no knowledge of relations.

Investigation in a New Direction—Realizing the necessity for a different kind of physical study of soils, the author in 1917 undertook a program of experimentation in a new direction, beginning with a study of the elastic properties of cohesionless sand. This work was practically isolated, and at the outset found little encouragement, most of the author's friends considering the efforts to be unpromising.

The method which finally led to a considerable degree of success consisted of reducing each problem to its simplest possible terms, and using the result of one test for establishing a tentative hypothesis as to the causes of the phenomena observed, which hypothesis could then be verified or disproved by further tests. No investigation was begun until the preceding one had been carried to some definite conclusion.

Proceeding step by step in this way, the author was able to trace the many physical properties of soils to four underlying factors: (1) Friction between the

grain surfaces, including initial friction; (2) viscosity of the capillary water; (3) surface tension of the capillary water; and (4) the influence of the width of voids on the physical properties of the water itself. It was found that the striking differences between cohesive and cohesionless soils could be fully explained in terms of these four physical factors, the intensity of their effects essentially depending upon the size and the shape of the soil particles.

Beginning in 1920 and continued within the last year, the author presented a summary of the action of these four physical factors and how in combination they produce the manifold physical properties of soils, in this journal.[1] A fuller account of the results of the investigation was published in book form a year ago.[2]

Application of Soil Mechanics—For practical application, soil mechanics must be applied in the first instance to the following objectives:

(A) *A theory of models* must be developed. Without such a theory no valid conclusions can be drawn from the results of loading tests on soils or from model tests (as on dams), with respect to full-sized structures.

(B) *Classification of soils* must be accomplished. It should be possible to determine and express quantitatively the relation between two apparently similar soils found in different localities.

(C) *Adequate design data* must be developed. The new science must guide the way toward obtaining all the required data for the economical design of structures consisting of soils or in contact with soils.

A Theory of Models

In modern engineering, theories of models are among the most important and most indispensable tools. To illustrate the necessity of such theories in earthwork engineering, the following example may serve:

The Foundation committee has designed a convenient apparatus for making loading tests on foundation strata, by means of which accurate load-settlement diagrams may be obtained. So far, however, no hint has been given as to how the resulting diagrams can be used. The ultimate bearing capacity of a clean, thoroughly compacted sand amounts to 0.29, 0.42, or 0.61 kg./cm.² (Strohschneider), according to whether the diameter of the loaded area is 0.8, 1.25, or 1.78 sq.cm., and it is known that a circular footing 1 m. in diameter resting on such sand could carry a load several times as great without objectionable settlement, so that it is clear that the bearing capacity of sand increases rapidly with size of loaded area. In the case of plastic clay, on the other hand, the bearing capacity is almost independent of the area of the loaded surface. Soils intermediate between sand and clay will have intermediate properties. Thus, in the absence of a theory of models no conclusions can be drawn from the results of a standard loading test unless a rule of interpretation is established for each of the many kinds of soil encountered.

Even with such rules we would still have the difficulty of identifying the soil at a particular foundation site with one of the standard soils for which the rules were derived. Non-homogeneous or stratified soil deposits further complicate the matter, and make a purely theoretical approach seem hopeless. Therefore the author believed it necessary to approach the theory of models by combined empirical and theoretical attack. In order to do this, each experiment, being in effect a model test, was analyzed theoretically to the point of making clear what physical factors were concerned and developing a formula expressing the law of the result as closely as possible. These formulas were then applied to deriving coefficients having known values for definite limiting conditions (as for plastic clay and for cohesionless sand) and intermediate values for intermediate conditions.

Bearing Capacity of Soil—Some of the results obtained by this course of procedure may be stated as follows: The bearing capacity of a soil depends on its specific gravity and density of structure, the smoothness of the grain surfaces, and the intensity of the capillary pressure, in other words its cohesion. The formulas derived for the bearing capacity of the ground contain as factors the specific gravity w of the soil, the intensity of the capillary pressure p_k, the radius of the loaded surface r, and the ratio t/r between depth of foundation t and radius of loaded surface.

In wet sand, capillary pressure is negligible compared with load stresses, and the ultimate bearing capacity increases directly as the radius r and also increases very rapidly with the depth ratio t/r. In plastic or semi-solid clay, however, the capillary pressure is very large, and the load stresses are negligible in comparison. Application of theory led to the realization that under such conditions the bearing capacity is almost independent of both radius and depth-ratio. Most actual cases will lie between these two limiting cases. For any given ratio p_k/w increasing the size of the loaded area will cause the case to approach the limiting condition of cohesionless sand.

While the theory so far developed accomplishes this much, and also is of assistance in devising special tests for cases of variable soil strata, it is by no means perfect, nor have the laboratory test methods yet been developed to their best form. The results demonstrate, however, that the combined theoretical and empirical method is effective. An interpretation of the results of loading tests is now possible, however much it may still need refining. This could not have been accomplished without laboratory discovery of the fact that the cohesion of clay is due to negative hydrostatic pressure of the capillary water; that the physical properties of capillary water contained in sub-microscopic voids differ from those of water contained in coarser materials; and that the internal friction of soils is a composite of several independent actions.

A theory of models also has other applications. For example, the test loading apparatus proposed by the Foundation committee[3] is intended to be applied to a loaded area 34.7 cm. in diameter, the surrounding ground being covered by a layer of sand 70 cm. deep. Here the ratio t/r is about 4. Looked at empirically, the sand fill is only a protecting layer, and it was actually so intended. Considering the situation theoretically, however, we see that the fill may be without effect on the result of the load test or it may multiply the bearing capacity several times, depending on whether the subsoil has large or very small cohesion. Because of this relation, we are bound to conclude that a loading test intended as a guide for the design of a sand foundation of shallow depth (t/r very small) should be so arranged that the ratio t/r in the test is also small, otherwise the test results may be misleading. The disturbing influence of the protective layer becomes insignificant, however, when the soil has high cohesion.

The lack of a theory of models is responsible for the fact that engineers hold curious and highly arbitrary views as to the interpretation of small-scale experiments on soil or soil structures. Investigators who undertake large-scale experiments and find their results differing widely from those of small-scale experiments commonly explain the disagreement by charging the small-scale experimenter with being unable to appreciate the importance of errors of observation. Such charges are manifestly unfair; the real trouble is the lack of a theory of models, which in turn is due to incomplete knowledge of the physical character of the phenomena involved. Every apparent contradiction between the results of small and of large-scale tests is a symptom of a specific deficiency of our insight. No problem in soil mechanics can be considered solved unless the investigator is in a position to explain fully the cause of such disagreements. For this reason small-scale experiments should be regarded an essential part of and a supplement to every large-scale investigation. Figures alone are practically worthless. What we need is relations.

Soil Classification

Every piece of actual construction work represents an expensive full-scale test. If we are in a position to identify accurately the soil encountered at construction, the value of the test results is inestimable. If, on the other hand, the soil remains unidentified, the same test is practically without any value, so far as future utilization of the test results is concerned. Hence, a method for soil identification is just as important as a theory of models.

It has been shown that variations in character of soils are merely the visible effect of corresponding variations in size and shape of the soil particles, in water content and in structure. If these four factors were known, it would be easy to identify soils. That the importance of these factors was well appreciated is shown by the fact that the Foundation committee's classification system takes them duly into account,[1] though unfortunately in a form not applicable to practical purposes. The properties of most soils depend on those of their smallest (so-called colloidal) particles, and one of the most important characteristics of these particles, the shape, is not susceptible of quantitative expression. Similar difficulties are met in dealing with another factor of outstanding importance, the chemical character of the adsorbed constituents. Due to the utter complexity of colloidal behavior the problem of comparing soils is in some respects like that of comparing the quality of two apples. To solve this problem according to the proposals of the Foundation committee would mean basing the comparison of the apples on the results of a physico-chemical analysis of their organic constituents. Realizing the doubtful benefit to be derived from so elaborate a procedure, the author proposed, symbolically speaking, the more direct method of biting into the apples and tasting them.

Basis of Classification—In the realm of soils, the taste of the apples corresponds to the common physical properties of soil (elasticity, cohesion, permeability and the like). Although these properties are numerous and each represents the combined effect of several causes, they offer the decisive advantage that they are susceptible of accurate numerical expression by coefficients and diagrams.

The accompanying table gives a complete list of these properties. In their totality these properties express every detail of the intricate properties of the soil constituents, somewhat as the elastic properties of a metal express the nature of its intricate molecular structure, or as a man's actions express his character.

Due to the great number of items, an exhaustive study of a soil sample cannot possibly be made except for scientific purposes. However, theory and experience show that the several properties listed in sections *B* and *C* of the table are all interrelated, although the precise nature of this relationship cannot yet be stated. A systematic experimental investigation is required to establish it. If 50 or more soils of widely different character should be tested according to the full schedule of the table, it might be expected that the assembled results would soon reveal the nature of the interdependence of the physical properties. With this knowledge the tests required for furnishing complete data on the character of a particular soil could be so simplified as to make them applicable to every earthwork reconnaissance—a future result likely to be of the utmost value. By a partial unraveling of these relationships I recently succeeded in simplifying the method for estimating the intensity of the capillary pressure acting at different depths of a mud deposit, to such an extent as to save nine-tenths of the time required for testing the drill samples.

This illustrates the gain that may be expected to result from the study of the relations between soil properties. Until these relations have been worked out, however, and test programs framed accordingly, it will be necessary to adapt any phase of soil tests to the particular practical objective of the work in hand.

Adequate Design Data

Design Equivalents—In soil mechanics as in other fields, ready calculation for purposes of approximate analysis and designing requires making use of simplified empirical assumptions. In hydraulics we assume water to be incompressible, and in mechanics of materials we use Hooke's law as if strictly true. Similarly, for soil mechanics we may assume as closely correct the following principles: (1) That the ratio between modulus of elasticity and internal pressure is constant; (2) that the relation between stress and strain, for direct compression with free lateral expansion is parabolic; and (3) that Darcy's law is valid. These assumptions are extremely simple, yet most practical problems are of such nature as to lead to intricate problems in the calculus of variations if an exact solution were attempted. Even if the stress-strain law were linear (Hooke's law) it would be exceedingly difficult to deal with a simple case such as stress distribution in a trapezoidal earth embankment, while with the parabolic law the problem becomes impossible. Obstacles of this kind make necessary a further process of simplification, using *design equivalents*, i.e., approximations similar to those utilized in other fields—as when a steel bridge is assumed to have frictionless joints—and subsequently making separate allowance for the approximation if necessary.

This method of equivalents may be applied as follows: In studying, say, the effect of a load on the supporting soil, or the swelling of clay after driving a tunnel through it, or the effect of time on pile friction, the first step is to study what physical factors are involved in the process, which is readily done if the physics of

soils is understood. Then an equivalent is devised, so as to represent an interaction of physical factors closely similar to the actual one, but yet simple enough for mathematical analysis. Formulas based on this equivalent give a clear conception of the relative influence of the several physical factors on the actual process, although they are strictly approximate ones.

To take an example, the equivalent for a circular foundation is a cylindrical soil core surrounded by a cylindrical soil ring. The top of the soil core corresponds to the loaded area. The loaded core presses laterally against the inside of the ring. The proper ratio between radius of core, height of core, and width of ring was estimated from the results obtained by photographing through a glass window the grain movements produced in soil by a superincumbent load. Formulas derived from such an equivalent agree better with test results than the formulas furnished by any other known method, and in addition give other valuable information, such as the influence of rate of load application on bearing capacity (in the case of wet clay), or the influence of the shape of the base (circular or rectangular) on the bearing capacity.

DATA REQUIRED FOR DESCRIBING PHYSICAL PROPERTIES OF HOMOGENEOUS SOILS

A—Material in Original Condition

1. Volume of voids, in per cent of total volume. For clayey soils in addition: microscopic structure (homogeneous, fissured, crumbly, etc.).
2. Moisture content, in per cent of weight of solid matter.

B—Modified Condition

3. Shape of grains (by microscope; include sketches where possible).
4. Specific gravity of grains (by pycnometer).
5a. (For plastic soils.) Limiting moisture content of liquid, plastic, semi-solid and solid consistencies.
5b. (For sand and granular soils.) Limiting void ratio of loosest and densest structure.
6. Coefficient of internal friction (minimum and maximum values determined at zero capillary pressure).
7. Lateral-pressure ratio in confined material (determined at zero capillary pressure).
8. (For plastic soils only.) Compressive strength of cube (dried at 100 deg. C.).

C—Diagram Characteristics

9. Uniformity (grain-size platted on semi-logarithmic chart).
10. Pressure-void diagram under continuous and cyclic loading.
11. (For plastic soils only.) Load-compression diagram for cubes of known water content under continuous and cyclic loading. (The diagrams 10 and 11 combined make it possible to compute the constants of elastic behavior.)
12. Permeability-void diagram.

Aside from its direct use in design, the method of equivalents will be of service in developing a classification of soils for purposes of practice. If the kind and relative importance of the physical factors governing a particular field phenomenon (as stability of slopes, or bearing capacity of ground) is known, a sample of soil may be tested in the laboratory for the particular coefficients which control these physical factors; then it may be concluded that every other soil having approximately the same values for these particular coefficients will behave similarly, so far as this phenomenon is concerned. The other physical factors can be disregarded, without affecting the value of the result.

Proper use of the method also helps to avoid unwarranted generalizations. Such generalizations have been particularly common in connection with the retaining-wall problem. Our classic earth-pressure theories are themselves mere design equivalents, in the sense just discussed; the sand is assumed to have a definite coefficient of internal friction, the effect of soil deformation on pressure distribution is neglected, etc., all of which assumptions are tolerable only for the purpose of the equivalent. Yet most of those who have dealt with retaining-wall experiments and analysis look upon these assumptions as rigidly correct, and in some cases resort to highly advanced mathematics to calculate the curved sliding plane of least resistance, or other details whose importance is negligible compared with the errors involved in neglecting the elastic effects. Again, the attempt has been made many times to apply Rankine's theory to determining bearing capacities, completely overlooking the fact that arching effects prevent the soil particles near the base of the load from yielding laterally.

Or, again, consider the striking remark made by a well-known authority in a recent discussion: "When an experimenter finds the angle of friction greater than the angle of repose, it at once condemns his results and shows that he is not well grounded in theory," which overlooks the radical difference between a change from motion to rest (which determines the angle of repose) or a change from rest to motion (which determines the internal friction). The fact is that the inclination of the surface on which a moving body comes to rest is invariably smaller than the angle at which a body at rest starts to move. If existing theories do not agree with such facts, the theories, not the facts, must be modified. But these examples are sufficient to show the danger of unwarranted generalization in soil mechanics. A proper study of equivalents will help to avoid this danger.

Further Developments

Engineering practice concerned with soils still depends, as did structural engineering a century or two ago, on the instinctive genius of a few individuals, and on the chance discovery and development of such individuals. Efficient and economical design is still largely a matter of hopeful anticipation rather than actual accomplishment. Moreover, whatever is accomplished in this field by the strength of individual genius remains merely an admirable monument and is of no lasting benefit because we remain powerless to identify the soils available for similar operations in two different localities, and lack methods for interpreting and applying the results of tests.

To improve this condition we need first of all a new generation of earthwork engineers, thoroughly familiar with the essentials of soil physics and trained from the very start for exhaustive analysis of the phenomena which are apt to occur in actual earthwork practice. Considering the amount and the quality of knowledge required for such activities, the courses offered by the colleges in engineering geology and in foundation engineering are obviously inadequate. On the other hand, it does not seem feasible to extend the scope of these courses without simultaneously reducing the time provided for other important subjects. Hence, the only appropriate measure would be to increase the number of traditional civil engineering options (sanitary engineering, structural engineering, etc.) by one. This new option (earthwork and foundation engineering) should include among others a more elaborate course in engineering geology and an elementary course in applied colloid chemistry.

The second essential requirement concerns the activities of engineers engaged in earthwork and foundation engineering. No satisfactory improvement is possible unless at least some of these engineers keep in permanent contact with the science of soil behavior, and try to apply the available information to their observa-

tions in the field. Progress in engineering essentially depends on the quality of the data published in professional papers, and thus far, due to inadequate discrimination between essentials and non-essentials, most of the published data have but little value.

Finally the profession needs at least one institute for scientific earthwork engineering, equipped with suitable laboratories and with a trained research staff, preferably incorporated in the organization of the department of civil engineering of some leading university. Besides serving educational and research purposes, such an institute would have two essential functions: to analyze and digest the results of important observations made and communicated by outside engineers, and to develop suitable field methods for preliminary work, based on scientific principles.

If development plans of this kind are followed out, soil mechanics may within a short time become an important factor of engineering science. In the absence of such conscious progress, the experiences of the present generation will be as valueless as those of the past, because of unintelligible records of the properties of soils.

[1]See an article by the author, "Old Earth Pressure Theories and New Test Results," published in *Engineering News-Record*, Sept. 30, 1920, p. 632; and the preceding articles of the present series, "Principles of Soil Mechanics," as follows: I—Phenomena of Cohesion of Clay, Nov. 5, 1925, p. 742; II—Compressive Strength of Clay, Nov. 12, p. 796; III—Determination of Permeability of Clay, Nov. 19, p. 832; IV—Settlement and Consolidation of Clay, Nov. 26, p. 874; V—Physical Differences Between Sand and Clay, Dec. 3, p. 912; VI—Elastic Behavior of Sand and Clay, Dec. 17, p. 987; and VII—Friction in Sand and Clay, Dec. 24, p. 1026.

[2]"Erdbaumechanik," by Charles Terzaghi (1925).

[3]Transactions, Am. Soc. C. E., 1920-1922.

[4]Transactions, Am. Soc. C. E., 1916, p. 351, Table II.

Silt and Channel Conditions in Colorado River Delta

How Deposition Contributes to Instability of Channel—Effect of Flood Scour on Levees—Pescadero Cut Dependent on Competent Engineering Maintenance—Present Condition Threatens Disastrous Results

By S. L. Rothery
Consulting Engineer, Los Angeles
Formerly Engineer with Imperial Irrigation District, Calexico

THAT the Colorado River needs a dam for storage and regulation of flow is illustrated in graphic manner by the photographic view, Fig. 1. Here is seen the dry, soil-filled bed of the lower river as it presents itself during the low-water periods, between floods. At a time like that illustrated, there is not enough water for irrigation, with the danger of vast losses, as in 1924, when 16 to 20 per cent of the cotton crop was lost on the reclaimed delta areas. And, as the river channel is vanished, the subsequent floods, which may attain volumes of 200,000 sec.-ft., create formidable dangers and present corresponding difficulties of control.

Silt Volume—Recurring soil-filling of its channel in low-water periods followed by deep scouring to more than 20 ft. below its normal bed along some meander where the rising flood happens to channelize, is a characteristic of the lower Colorado, and involves soil transportation on a huge scale at each scouring. The main channel being 800 to 1,200 ft. wide, a scour depth to 20 ft. transports more than 300 million cubic yards or 186,000 acre-feet of soil to the delta for each 100-mile length of river in flood. This may appear an excessive estimate, for it is not possible to know the area of a scoured channel during floods. However, the soil transported in suspension past Yuma for the decade 1915-1924 has been measured as 1,051,350 acre-feet. Some of this latter amount is undoubtedly bed soil carried into suspension with increasing discharges, but the writer believes it is a portion only, for the suspended soil is measured continuously through the year, while scoured channels happen during periods of a few days to a few weeks only, when hundreds of miles of channel are scoured to great depth at the same time, and the former vast volumes of bed soil are transported in these short periods.

A volume of 1,051,350 acre-feet, the 10-year silt flow at Yuma, would build a railroad embankment around the world at the equator, which embankment would have a height of 12 ft., a base width of 50 ft. and a top width of 10 ft.; or it would occupy a volume equivalent to that of a prism a mile high covering 64 city

FIG. 1—DRY, SOIL-FILLED BED OF LOWER COLORADO RIVER
Every high flood scours the channel 20 to 40 ft. deep and transports the soil (186,000 acre-feet for each 100-mile length) to the delta.

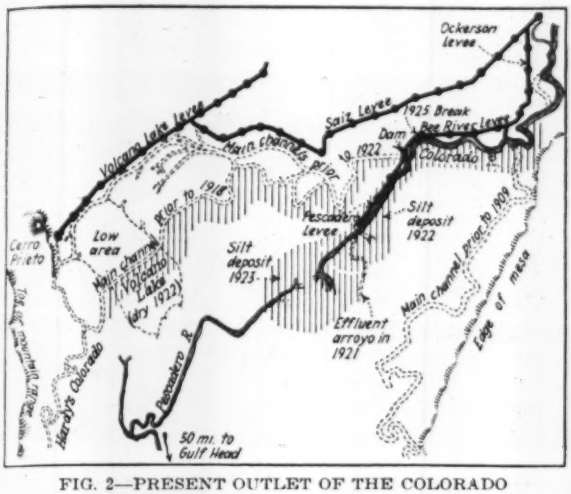


FIG. 2—PRESENT OUTLET OF THE COLORADO
A few of the former channels are indicated by dash lines.

SILT VOLUME OF COLORADO RIVER

Amount transported in suspension past Yuma, 1915-1924.

Year	Silt Volume, Acre-Feet	Year	Silt Volume, Acre-Feet
1915	107,880	1921	111,894
1916	204,834	1922	63,964
1917	89,062	1923	125,145
1918	59,831	1924	95,407
1919	68,333		
1920	125,000	For decade	1,051,350

blocks 300 ft. square, each block being separated by streets 80 ft. wide. The same prism, but 528 ft. high, would represent the average yearly equivalent volume transported in suspension.

Channel Changes — Formerly the river had about 2,500 sq.mi. of delta upon which it wandered and distributed its soil burden, and great changes of location were not of consequence. Since 1908 the surplus discharge—nine-tenths to five-sixths of the yearly flow of the river—has been kept to the south of an extensive flood-protective levee system which has been developed

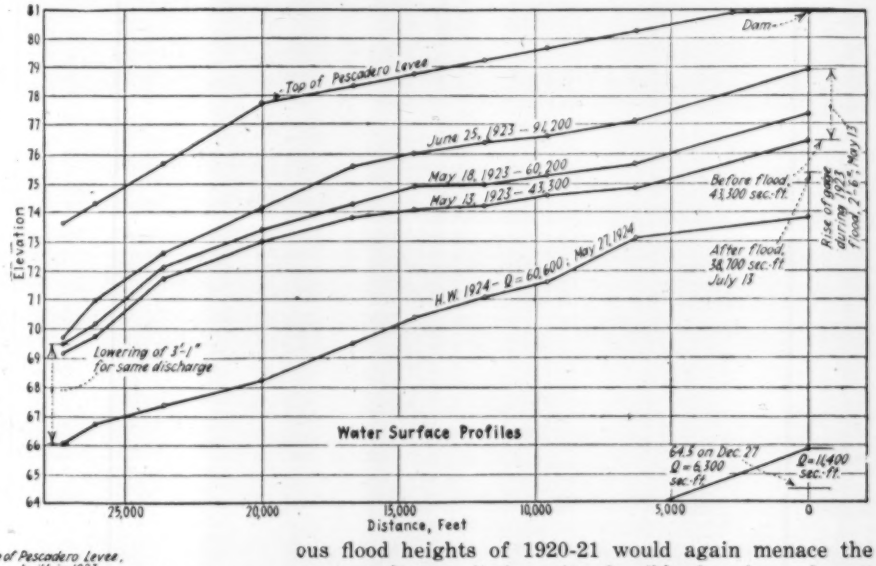
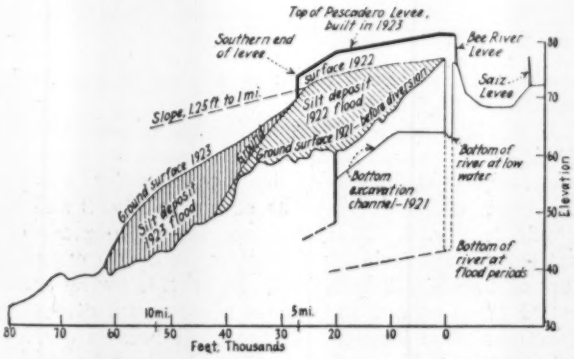


FIG. 3—LONGITUDINAL PROFILE OF PESCADERO CHANNEL



in Baja California, Mexico, restricting the former large dumping ground to about 700 sq.mi. On this diminished area the development of unstable flow conditions has been hastened by the magnitude of soil deposition, and since 1908 the river has abandoned more than 130 miles of main channel for entirely different directions of flow.

In 1909 the final 60 miles of a general southerly course to the gulf were abandoned for a westerly and thence a southerly course which was 30 miles distant from the former route. In 1918, 15 miles of the westerly portion was forgotten for a southwesterly direction against the Volcano Lake levee. Again, in 1922 diversion operations forced a general southwesterly course towards the gulf and the abandonment of 55 miles of its main channel of more recent years. Portions of these abandoned channels are shown in Fig. 2.

The Pescadero Diversion—The diversion last mentioned, into the Pescadero country, has now been functioning four years, and while there is ample low area available for years of soil deposition before the hazardous flood heights of 1920-21 would again menace the western levees, it is quite feasible in view of past channel changes and of the significant break and sloughing along Bee River levee in 1925, for a disastrous breach in these river levees to happen on a rising flood. In that event the diverted course may likewise become abandoned and silted full. This would

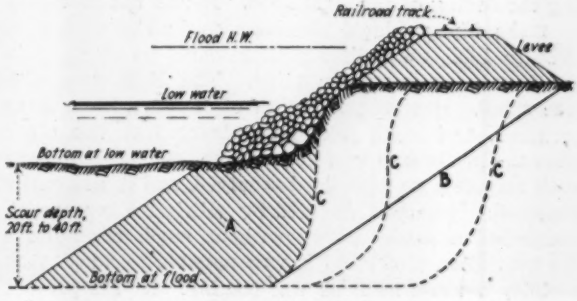


FIG. 4—LEVEE SCOUR AND PROTECTION
Typical sketch. Deep flood scour will undercut levee along a convex curve in a few hours. Unless enough large-size quarry stone is quickly at hand to fill several hundred lineal feet of such a large area as *A*, the levee will slough into the river along line *B* or will be undercut as the side of the channel *C* advances with erosion.

FIGS. 5 TO 9—SOME DELTA CONDITIONS PORTRAYED BY PHOTOGRAPH

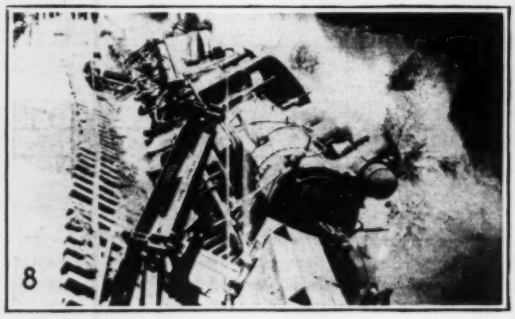
Fig. 5—River in flood, seen from airplane. Country submerged from river in background to levee line in foreground. Trees whose tops can be seen along river bank help to retard overflow and cause rapid deposition.

Fig. 6—Flood flow through small break in levee.

Fig. 7—Trainload of rock going to threatened levee.

Fig. 8—Maintenance train overturned on saturated levee. Such accidents are strong reasons against relying too much on emergency measures to deal with flood dangers.

Fig. 9—Pescadero Channel before diversion. This is the channel into which the diversion of 1922 was directed. It may be seen in cross-section on the profile, Fig. 3. In the first season's flood after the diversion the beaver dam seen in the view was buried under 25 ft. of alluvium. The channel, 12 ft. deep, was filled and an additional depth of 13 ft. of soil was spread over the land surface.



be a reversion to 1921 conditions, and the asset of the Pescadero diversion would be entirely lost.

No high flood occurred in 1924. Moreover, the low-water surface had receded before the flood season, during the falling of the 1923 flood. It was not necessary to make topographical observations in the winter of 1924 to 1925, other than such as could be obtained in the course of inspection trips. The 1925 flood also was small. However, a breach of the Bee River levee occurred, and much sloughing occurred at two other places. The breach was not serious, as (in accordance with forecast) no high flood occurred, and it was easily shooflied. Economy, dictated by political management, prescribed a get-by policy without much expenditure for protective work, and it was possible to adopt such a policy because no flood was coming. At the present time, however, it is important that surveys should be made to give data on the past two years' deposition along the Pescadero diversion, and to arrange for the protective and maintenance measures without which the works in their present condition will not be capable of resisting a high flood.

If the Pescadero diversion channel is once lost, a diversion by erosion along the same route could not be re-accomplished, because of the huge silt deposits along the upper diverted course, as shown in Fig. 2 and in profile in Fig. 3. Short distance and steep grade do not exist again from the Pescadero dam. An expenditure of a million dollars would not provide for an excavation large enough to promise a successful diversion from the same location. If the river became divided into two channels, one flowing through a breached levee, the diminishing flow to low-water conditions would soil-fill one of them, and the chances are that the diverted course into the Pescadero country would be the channel that would be lost.

No Engineering Supervision—Engineering supervision and direction on the delta have now been dispensed with, and a policy is substituted of divided responsibility without engineers, that will mean no direction

based on a sound engineering knowledge of changing topography, of soil-filled meanders and of new channel positions. The needed knowledge can be obtained only by consistent observation of the river's behavior to foretell points of scour danger along the levees, where perhaps 30 to 50 cu.yd. of rock revetment per lineal foot of levee may not be too much (as illustrated in Fig. 4) and by consistent recording of data on gage heights, levee freeboard, and river slopes, for men who know their value.

Physical changes occur so fast on the delta that such data put in graphical form present important comparative pictures for study. Analyzing these in the light of the experience of past years and previous floods, and long-time personal familiarity with the rivers, furnishes sound knowledge that diminishes much of the oft-quoted uncertainty expected of the lower Colorado. But the repeated dismissals of engineers as soon as they have become educated to delta conditions is a menace to the security that is apparently afforded by the protective levees. Engineering direction of that small but essential work to be done to prevent too much reliance on emergency measures during critical times, is consistently necessary to safeguard an accomplished work and thereby forestall the unexpected happening that will be again the excuse of the politician, if a reversion to 1921 conditions occurs.

Should such reversion happen in the next decade it would be a calamity, inexcusable from an engineering viewpoint. That it can happen is beyond doubt, for big floods with subsequent low water flows are due, and will undoubtedly occur several times before relief can be provided by constructed storage dams. And if a policy is adhered to of running a great ship without a captain—in this case an able engineer with the added experience obtained in the laboratory of the Colorado River Delta—the calamity *will* happen.

Low Concrete Strengths Found in Tests of Ribbed Floors

TESTS of ribbed floors of reinforced concrete and hollow tile recently carried out at the Bureau of Standards, Washington, and reported in Technologic Paper 291 by D. E. Parsons and A. H. Stang, produced some surprisingly low concrete strength figures.

The tests as a whole, which were made in co-operation with the Hollow Building Tile Association and had the object of determining whether the tile between the ribs of such floors contribute to the bending strength of the floor, revealed that the tile add considerably to the strength. The strength increment substantially corresponds to monolithic co-operation of tile and concrete, taking due account of the modulus of elasticity of the tile (in some of the tests this was lower than that of the concrete and in other tests higher). All of the beams were 8 in. deep. Some had 4-in. ribs, a 2-in. slab, and 6x12-in. tile, while others had no slab but consisted of 8x12-in. tile between 4-in. ribs, 8 in. deep. In both types of floor the bond between the concrete and the tile was sufficient to develop monolithic action, both at working stresses and at ultimate loads. This was shown both by the strength and deflection values and the manner of cracking of the beams at failure, the cracks going continuously through tile and concrete ribs. The tests all were based on direct comparison of composite tile-and-concrete beams with concrete beams containing no tile, and the results are therefore independent of the strength of the concrete. Cylinder and prism tests of the concrete, made upon specimens molded at the time the beams were poured, show variable and in general very low strengths of concrete.

A 1:2:4 mixture of concrete was used, made with Potomac River sand and ¾-in. Potomac River gravel; the mixing was done in a drum mixer. Two compression specimens were made for each beam, one a 6x12-in. cylinder cast in a standard steel mold, the other an 8x8x12-in. prism molded in a form of which two sides were made of oiled wood and the other side of either tile or wood, according to which material was in contact with the concrete in the corresponding beam. All these compression specimens were stored with the beams within 48 hr. from the time they were cast, and were tested at the same age as the beams or 102 to 106 days.

COMPRESSIVE TESTS OF 1:2:4 CONCRETE CYLINDERS AND PRISMS
(Age at test from 102 to 106 days. Storage the same as for corresponding beams)

Number	Date Poured	6x12 in. Cylinders: Compressive Strength	6x12 in. Cylinders: Modulus of Elasticity	8 x 8 x 12 in. Prisms: Compression Strength	8 x 8 x 12 in. Prisms: Modulus of Elasticity	Form Material
	1923	Lb.-in.²	Lb.-in.²	Lb.-in.²	Lb.-in.²	
E-2	Dec. 7	2,230	2,180,000	2,730	.....	Shale tiles
D-2	7	1,950	1,910,000	2,120	.....	Fire-clay tiles
D-3	7	1,530	1,770,000	2,450	.....	Fire-clay tiles
G-2	7	1,060	1,720,000	2,530	.....	Fire-clay tiles
H-3	7	2,050	2,020,000	3,310	.....	Surface-clay tiles
Average	.....	1,760	.....	.....	.....	
E-3	14	990	1,860,000	1,560	2,540,000	Shale tiles
A-2	14	850	1,450,000	1,450	1,650,000	Wood
B-3	14	1,100	1,780,000	1,490	1,210,000	Wood
N-1	14	880	1,540,000	1,800	2,220,000	Fire-clay tiles
K-1	14	1,130	2,160,000	1,570	2,150,000	Fire-clay tiles
Average	.....	990	.....	.....	.....	
N-2	21	1,260	1,780,000	2,210	2,450,000	Fire-clay tiles
M-1	21	850	970,000	1,850	1,900,000	Fire-clay tiles
O-1	21	1,220	2,140,000	2,640	3,300,000	Surface-clay tiles
L-1	21	1,310	2,440,000	2,140	2,190,000	Shale tiles
Extra	21	1,250	1,700,000	1,700	1,710,000	Wood
Average	.....	1,180	.....	.....	.....	
C-1	28	1,140	1,700,000	3,080	3,390,000	Surface-clay tiles
C-2	28	1,030	1,670,000	2,370	2,460,000	Surface-clay tiles
F-1	28	1,010	1,650,000	2,330	3,060,000	Surface-clay tiles
G-3	28	1,380	2,320,000	1,790	3,420,000	Fire-clay tiles
L-2	28	1,360	1,740,000	1,960	2,500,000	Shale tiles
Average	.....	1,180	.....	.....	.....	
	1924					
C-3	Jan. 4	1,100	1,670,000	1,240	2,040,000	Surface-clay tiles
F-2	4	990	1,380,000	1,780	2,550,000	Surface-clay tiles
J-1	4	880	1,430,000	1,330	2,080,000	Surface-clay tiles
K-2	4	820	1,370,000	1,430	2,560,000	Fire-clay tiles
M-2	4	1,030	1,410,000	1,240	2,020,000	Fire-clay tiles
Average	.....	970	.....	.....	.....	
F-3	11	450	990,000	1,440	2,040,000	Surface-clay tiles
J-2	11	530	900,000	1,150	2,020,000	Surface-clay tiles
O-2	11	720	1,240,000	1,240	2,080,000	Surface-clay tiles
K-3	11	560	1,050,000	1,030	1,940,000	Fire-clay tiles
M-3	11	470	970,000	1,330	1,960,000	Fire-clay tiles
I-1	11	660	1,050,000	1,020	.....	Wood
Average	.....	570	.....	.....	.....	
N-3	18	1,470	1,740,000	1,570	1,670,000	Fire-clay tiles
J-3	18	950	1,750,000	1,500	2,180,000	Surface-clay tiles
O-3	18	1,070	1,480,000	1,650	2,020,000	Surface-clay tiles
L-3	18	1,140	2,000,000	1,730	2,600,000	Shale tiles
Extra	18	1,170	1,960,000	1,430	1,060,000	Wood
Average	.....	1,160	.....	.....	.....	

From the attached table of results it appears that individual specimens ranged in compressive strength from 450 to 2,230 lb. per sq.in. in the case of the cylinders and from 1,020 to 3,310 lb. per sq.in. in the case of the prisms. The prisms consistently showed greater strength than the cylinders, by 50 per cent or more. Nevertheless, one group of six prisms averaged only 1,200 lb. per sq.in. compressive strength, or about half the usual compressive strength of concrete of this mix and age. The corresponding cylinders averaged only 570 lb. per sq.in., ranging from 450 to 720. The general average for the 36 cylinder specimens was 1,120 lb. per sq.in.; that of the 36 prisms 1,810 lb.

Road Progress in the South Atlantic and Gulf States

South Carolina	Louisiana
Georgia	Florida
Alabama	Mississippi

An Editorial Field Study of State Road Organization, Policies and Plans; the Trend of Highway Thinking; the Status of State Control; and the Progress in Engineering and Construction in Six Southern States

By John W. Shaver
Assistant Editor, *Engineering News-Record*

IV—Mississippi

OF ALL the highway departments in the Southern states that in Mississippi has had the most difficulty in establishing itself as the proper agency through which should be handled road construction and maintenance of a state-wide character. The highway commission is still fighting for proper recognition and construction funds. Since its inception in 1916 it has had little money with which to build highways, and it has always been the object of suspicion on the part of those state legislators who hearken so closely to the demands of local self-government. The highway commission today is just getting to the point where it has convinced the general public that what it is trying to do is sober and efficient road-making, but it yet has to convince the state legislature. However, it is hoped that at the 1926 session of that body at least proper authority will be given the commission, and a small construction fund. But heretofore any attempt to secure the sort of legislation that makes for the construction and maintenance of a state highway system that will meet traffic needs has been so thwarted that what legislation has been enacted has left county authority more or less supreme.

The reasons for such a condition are obvious. Mississippi has always been one of the staunchest of supporters of that form of government which meant local political control. It is difficult to override such a theory without any practical application. But when the theory is supplemented by the vesting in a state body power for complete administration of funds that are supplied by the counties themselves, the task is doubly difficult. In the minds of the local politicians remains the old belief that old ways are better, and that perhaps the younger generation of ideas will wear out with little use. The job of the state highway engineer has, therefore, been one of education and missionary work. It has been ambassadorial rather than technical. Two more years will doubtless make of the state highway commission an organization that can devote its whole time to the development of a road system that does need a lot of money spent upon it. Just what the vicissitudes have been can best be learned by reviewing the commission's history. In the beginning let it be said that the commission has been the object of almost continuous legislative scrutiny, but investigators have almost without exception left the inquiries boosters for the state highway commission and its general program.

Organization and Growth — The Mississippi State Highway Department was organized for the express purpose of administering federal-aid highway funds. It was authorized by the legislature of 1916 and the act creating the department became a law March 29, 1916. The first highway act authorized the appointment of a three-man commission, and the first meeting of that commission was held April 14, 1916. The commission named Xavier A. Kramer, of Magnolia, Miss., as the first state highway engineer, to serve for a three-year term. Mr. Kramer organized the engineering force of the department, which was small, owing to the small appropriation which was allowed by the legislature.

The first appropriation for the support of state highway forces was $6,500 for each of the years 1916 and 1917 and $10,000 for each of the next two years. Under these appropriations the State Highway Department proceeded to make surveys, prepare plans and let to contract a number of federal-aid projects. The cost of construction was borne by the counties in which the projects were located. Due to the small number of engineers in the department considerable difficulty was encountered in getting adequate supervision of work and proper administration of federal-aid funds. The commission, however, took up all of the funds the federal government allotted.

Realizing the difficulties the highway department was facing the legislature increased the annual appropriation in 1920 to $100,000. During the same session of the legislature the highway act was amended so that the personnel of the commission was changed to eight instead of three members, one member to be elected from each of the congressional districts in the state. Mr. Kramer's term had expired but he was continued as state highway engineer by the new commission, until such time as a successor could be appointed. H. C. Dietzer was appointed in July of 1920 and has held the office continuously since that time.

Staff Enlarged—With the increased appropriation the engineering forces were enlarged and put in a position properly to administer federal aid and supervise construction. But the actual construction funds came from the counties. The 1920 legislature also gave the state highway department 50 per cent of the motor-vehicle fees, and 40 per cent of a one-cent gasoline tax, the latter to be used for maintenance purposes. A large percentage of the former was also needed for maintenance. In addition to maintenance the license fees were used to construct short connecting gaps on the principal routes, and major bridge projects. A little of it was used to supplement county contributions toward federal aid. With such an organization the state supervised the construction of about $15,000,000 worth of work, almost half of which was federal-aid money. The main routes of travel were graveled, though in many places (particularly in the poorer counties) roads existed that were impassable during wet weather.

The legislature of 1924 made other changes in the highway law. It designated by route number and location, the roads that were to comprise the state highway system. Theretofore no designation had been made, though most of the construction work had been on the intercounty-seat highways and main market roads. That session of the legislature also repealed the provision which allotted a flat appropriation of $100,000 yearly and substituted one, which provides about the same amount, whereby the highway department should receive the profit on the sale of automobile license tags. It also gave the state highway department half of the gasoline tax that had been raised to three cents a gallon, but it ordered that the entire fund derived from motor vehicle taxation be returned to the counties. As the profit from the license tags was about sufficient to operate the highway department, and as the gas tax had to be used for maintenance, there was still no provision made for a construction fund. And the highway commission has none today, although it is hoped the 1926 legislature will give it one.

Powers Still Limited—It would seem that, though the counties do provide the money for construction, the state highway department would be able to wield supervisory power over all roads on the state system. But it still exists solely for the administration of federal-aid funds and maintenance. The county may build a link on a state highway that does not participate of federal aid, and the state has no control over its construction. That link may connect two federal-aid projects. The only power the state has in the matter is one of veto. It may reject for maintenance any such road that the county builds which does not meet certain standards of quality and specifications. That part of the 1924 law covering the duties of state and county on the state roads follows:

The roads above set out, constituting the state highway system, shall be maintained by the state highway department and no county shall be required to pay from their own county funds or road funds any part of the cost of maintenance of any road within such county, which may be taken over by the state highway department for maintenance, and the said state highway department is hereby vested with full authority and jurisdiction over the same, so far as their maintenance is concerned; provided, however, that before any link of said highway system is placed under the control and supervision of said state highway department, the said link of road shall be constructed by the county where the same is located and at the expense of such county. And when the board of supervisors of any county shall construct a highway from one point to another designated above (the designated system) which comes up to the standard required and tenders the same to the highway department, said highway shall be taken over and maintained by said highway department. That on request from the board of supervisors of any county, the state highway department shall file with such board a copy of its general specifications covering the various types of improvement to be made by such counties to bring state roads to a general standard, recommending the type that should be employed, and the width and character of the improvement to be made, consideration being given to present and reasonably probable future traffic needs, on each highway. All such highways shall be accepted by the state highway department in writing and such writing spread upon the minutes of the board of supervisors of said county, and an order entered by said board authorizing the state highway department to have supervision over and control of the maintenance of such road or highway. But, if after said highway department has been vested with full authority and jurisdiction over any road as provided for herein, it shall appear to said highway department that the cost of maintenance thereof will be decreased materially by any additional construction or enlargement of said road said additional construction and enlargement . . . may be done by said highway department and said road thereafter maintained by said highway department, and the cost thereof paid for out of the funds provided in this act for the cost of maintenance.

Thus it is seen that the state has little control over construction types that are not to be used on projects participating in federal aid. As a matter of fact the difficulties extend further. At the present writing (November) the state highway department and two counties are at a deadlock over two projects which have been approved for federal aid. The difficulty is over location. The county holds out for its location and the state believes another is best. The probable result will be that the county will forego federal aid and build a lower type road on its own location. That will doubtless mean traveling over a fair-weather route instead of a year-round road.

Some road work of course had been done by the counties prior to the passage of the Federal-Aid Act, but the construction was spotty. Gravel was usually laid on whatever road surface had been worn, with little subgrade treatment. Such roads took little heed of alignment, section or grade. In numerous instances the state highway department has had to relocate roads that are to be constructed with federal funds participating. In the reconstruction of the Jackson-Vicksburg highway, for instance, a project that is soon to be paved, probably 90 per cent of the existing alignment will be abandoned.

Attitude Toward Federal Aid — Like most of the Southern states Mississippi is a fervent champion of federal aid. It is said that had federal funds been withdrawn from the state two years ago it would have killed the state highway department. Should they be withdrawn early next year it would be crippled, but probably not killed. And if the federal government offers financial aid for two years more it is probable that the loss of revenue alone will be felt. Because of the fact the Bureau of Public Roads has been strict in its requirements as to alignment, grade and section, before a road would participate in the federal funds, the Mississippi counties have been accustomed to a supervisory control of types and method of construction. They have also shown some respect for efficient technical control, even though the expense has been greater than they have been used to. Therefore, the Mississippi Highway Department feels that the greatest good that federal aid has done has been in the strict technical supervision which the Bureau of Public Roads has wielded.

All of the work in Mississippi is done by contract. The counties actually sign the construction contracts and then turn the money over to the state treasurer, who meets the total payment for the work. If there is any money left after the highway is accepted it is returned to the county making the original payment. The state supervises the construction and, because it is a federal-aid project, takes over the road for maintenance.

Construction Types—Practically all of the construction is gravel. In about 5,500 miles in the state system, 3,500 miles of which are upon the important routes, the state maintains 2,500 miles and 90 per cent of that is surfaced. There are only about 160 miles of hard surfacing in the state on state routes, and all but about 20 miles is reinforced concrete, the other being divided between asphalt types and brick.

Gravel roads are built either of material taken from

roadside pits or of imported material. If gravel is locally available it is furnished by the county, and the contractor bids only on grading and placing. That system also holds for imported material, except for the fact that bids are taken on supply of gravel at the same time they are taken on grading and placing. In neither instance does the contractor supply the gravel, unless he makes two bids or a combination one.

The gravel used must conform to the specifications which require that 100 per cent pass a 3-in. screen. Of this between 45 and 75 must be retained on a ¼-in. screen. From 15 to 75 per cent of that retained on a ¼-in. screen shall be retained on a 1-in. screen and of that passing a ¼-in. screen from 15 to 35 per cent must pass a 100-mesh. Though this gives a good grade of material it is believed that the specifications can be somewhat improved.

In construction, the subgrade is first brought to proper cross-section and a trench prepared by plowing or machining the dirt to the side of the road. At the edge of this trench boards are placed in order to secure the proper depth and alignment. The boards are so placed that their top edge shall conform to the grade of the finished road, and they are kept in place at least 200 ft. ahead of gravel placement. The subgrade between the boards is machined and rolled and kept in proper shape. The boards are removed after the gravel is spread. The road is then rolled until there is no sign of waving in front of the roller. Gravel is spread to a uniform thickness of 8 in. measured loose. The shoulders are well banked with earth so that when the sideboards are pulled the gravel edges will not ravel.

The reinforced-concrete road put down is what is called the Maricopa section: 9 in. thick at the edges and 6 in. thick at the center. Shear rods along the edges constitute the reinforcing, and expansion joints are laid from 30 to 50 ft. apart, with slabs doweled. The asphalt types include bitulithic, rock, modified topeka and sheet, but of all these there are only about ten miles.

Personnel—Though the engineering organization is not so large as is found in other Southern states, it is of a flexible character and could be made to carry on an extensive construction program with few changes. The state highway engineer himself is the real executive head of the commission, acting for it when it is not in session, though the commission has a chairman. Directly under him are the office engineer, construction engineer, maintenance engineer and bridge engineer. The construction engineer is really a supervisor of design, as he does not superintend construction, that being left to the district engineers of whom there are seven. There are eight construction districts but the maintenance engineer handles the eighth district in addition to his other duties. All construction and maintenance are supervised directly by the district engineers. Project engineers, who are really resident engineers, operate under the district engineers.

Maintenance Practice — Maintenance is mostly by patrol, though there are a few heavy-duty gangs that supplement the patrolmen's work. Patrols are of two kinds: Man and team, and motorized units. It is the former that the heavy-duty gangs supplement mostly, for the equipment that a team can draw is too light for heavy ditch work. The teams cover from seven to ten miles of road and the motor units as high as twenty miles.

There is one detail of maintenance which is unusual The highway department can take over a road for maintenance without taking any of the bridges on it. That is done because so far the state has had little money with which to recondition bridges, all of that work, except on federal-aid projects, being done by the county. Bridges are accepted for maintenance only when they meet the standards imposed by the department. This covers all drainage structures, though most of the drainage structures on any road are culverts which the state really is able to install. It is estimated that to rebuild all of the bridges on state highways alone would cost about $25,000,000 if creosoted timber were used and about three times that amount for concrete structures. To rebuild structures that might participate in federal aid would cost $15,000,000 and $40,000,000, respectively.

The state highway department has all sorts of subgrade to deal with. In the delta region it is mostly silt in depths up to 100 ft. In the southern part of the state along the coast there is a sand-clay which makes an excellent natural road with little preparation. In the central section is found a very heavy clay with lime content that makes the worst sort of base for surfacing, and in the northeastern part there is a heavy black soil with a clay and lime content, with some outcropping of limestone. Such a wide variety of subgrade conditions must, when the state has funds with which to pave, become a subject for grave study. At the present time, however, gravel is spread on all of them.

Mississippi has had a little difficulty, as has Georgia, with contractors who bid too low. Several recent jobs have been completed by bonding companies. This is mostly for paving work. The gravel surfacing is something the contractors are better able to bid upon, but on concrete the total cost has run as low as $22,000 per mile, with the average around $25,000. It is apparent that either competition is keen or contractors are unfamiliar with the type of construction. At any rate low-bidding has cost the state inconvenience and some popular support. Gravel costs, for grading and draining, from $4,000 to $12,000 per mile, with the surfacing running from $2,000 to $7,500, making the average for the completed road between $12,000 and $15,000 per mile.

The Mississippi Highway Department is fighting hard for just recognition. It has overcome already a number of obstacles, but legislation is needed that will put adequate funds at its disposal and give it a free hand in the selection of location, alignment and type of highways that shall comprise the state system. Its problems are those of the pioneer. It is not so much concerned now with highway research as with saving an investment and getting on its feet. It is not a state from which one can get new ideas on design. But one can get a certain inspiration for its dogged attempt to educate the people and the legislature to the necessity of an adequate system of state-owned and controlled highways.

4,000 Tons of Sludge Towed to Sea Weekly

Treatment of the sewage of the Passaic Valley Sewerage District by sedimentation is now resulting in the production of 4,000 tons per week of wet sludge from 100 m.g.d. of sewage. The sludge is being barged to sea under a contract providing for dumping as far out as 10 miles from the Scotland light ship, at a price of 37c. per ton. J. Ralph Vanduyne, Newark, N. J., is chief engineer of the Passaic Valley Sewerage Commission

Engineering in the Small City

A series of notes from the field on the Problems and Practices of the Municipal Engineer in the Cities Below 100,000

The Editor Visits the City Engineer

Snapshots of Our Representative Small Cities

BUSINESS SECTION, MONTGOMERY, ALA.

Montgomery, Ala.

STATE CAPITAL OF ALABAMA and county seat of Montgomery County. Situated 180 miles northeast of Mobile, on the Alabama River at the head of navigation. Montgomery is an important commercial and railroad center distributing materials and manufactured products in all directions. The accessibility of timber and of coal and iron deposits has aided in the development of its many industries; there are railroad shops and factories of general and southern products. In 1540 De Soto discovered an Indian village near the present site of the city; settlers founded the place as New Philadelphia in 1817, it was named Montgomery in 1819, and in 1847 supplanted Tuscaloosa as state capital. For a time during the Civil War it was the capital of the Confederacy. Pop. 1920, 22,000.

THERE is scarcely a detail connected with the physical property of the city of Montgomery over which the city engineer has no control. Exceptions are equipment purchased for the police and fire departments. All the rest of the city's equipment is either purchased direct and operated by the city engineer, or bills for purchase must clear through him. He even has to okay bills for meat bought for the bear at the city park. And until recently he has exercised a sort of paternal watchfulness over the city's inhabitants, though, as a matter of fact, the shouldering of many incidental duties has not been of his volition. When the city engineer gets a telephone call that the grass in a backyard of some property owner needs cutting and that householder cannot find labor himself, it has been considered proper that the city engineer should get the labor. The obvious result of such responsibility has been to involve the department in small details from which, under the present régime, it is rapidly getting away. But his duties are still manifold.

Montgomery has the commission form of government. Three commissioners are elected, one of whom is the mayor, the others being the commissioner of fire and police, and of water supply and sanitation. The mayor is really the commissioner of streets. It may happen that the commissioners will change about even during their tenure.

The city engineer is appointed by the commissioners, and he in turn appoints his own engineering staff. The staff consists in three principal assistants (one of them called the assistant engineer) and subordinate office and field forces. Incidentally there are two rodmen, both colored, who have been with the city engineering department practically since its origin as a well-constituted office; the older in service is finishing his forty-first year. This colored rodman took great pride in recalling having piloted the late Rudolph Hering about Montgomery when the latter was in that city designing the first sanitary sewer which the city built.

Work in the engineering department is so divided that the assistant city engineer has charge of paving work, one of the others has sewers as his principal responsibility, and the third has charge of the office work. Naturally there is an interchange of duties where the occasion demands it. The assistant city engineer is also the city building inspector, by virtue of the engineering office he holds. There is another responsibility which the present city engineer has that is unusual. There is no statutory reason for his being so, but it so happens that J. M. Garrett, present city engineer, is chairman of the building committee of the school board, and by virtue of that office has something to say in the matter of school construction. Mr. Garrett has been in private engineering work in Alabama for many years, and has held the office of city engineer but a year. He and his son (and by the way all three of the principal assistants to Mr. Garrett are graduate engineers and young men) were among the project engineers employed by the state highway department before that organization had its own engineering force. They furnished the plans and supervised the construction of about the first eight of the federal-aid projects the state built. With such a regular staff as a nucleus the engineering organization is expanded as the work at hand requires. The appropriations allowed for engineering expense are liberal (amounting yearly to about $18,000 for salaries and office expense alone) and consequently there is afforded an elasticity which makes for competent engineering work.

Montgomery is not flat, so its drainage, both for storm water and sanitary sewage, is comparatively easy to accomplish. The presence in the vicinity of a very good grade of clay-gravel makes the building of earth roads inexpensive. Its engineering problems are not, therefore, so much of design as of operation.

Funds for carrying on public works construction are secured mostly through bond issues refunded by assessments upon the property benefited by the improvement. The entire cost of paving streets and intersections, curbs and gutters and sidewalks, is borne by the abutting property. Assessment of the intersection cost is made upon each side of the street in the four directions from the intersection, but only half the length of the street so affected. One-eighth of the intersection cost is pro-rated on a front-foot basis on each such half-block. As in other cities following this system of assessment, questions often arise from property owners when they get to comparing notes, on why there is such a variation in assessments for the same intersection, but it is due mainly to the fact that street lengths vary widely.

Though the city has the right to pave any street that it considers necessary, if more than 50 per cent of the property owners to be benefited by the improvement object to it, the street is usually held over for improvement until more favorable action can be secured. Payment for intersection paving formerly was borne by the four corner lots, but that system was not followed long, its injustice being obvious. Assessments are also made against abutting property to carry the cost of sanitary sewer construction.

In storm sewer construction (the sewerage facilities

are separate) the city has been following a plan of forming drainage areas, and assessing the cost of all storm sewer work within that area to the property drained. The cost is shared by all property on a front-foot basis, no matter how far from the work being done the lot may actually lie. The total cost of the work is figured, the total length of all lot frontages in the drainage district, and the per front foot cost found and the total cost per lot adjusted. It may happen that one man will own twenty or thirty lots in the territory so benefited, but an assessment is made for each of the lots. The city is now figuring on the construction of a sanitary outfall which is to cost about $150,000. It is estimated that at least 7,000 property owners will be benefited by the enlarged drainage facilities. Each of these will be assessed for the improvement, share and share alike, depending upon the lot frontage he owns. This improvement will entail the making of more than 8,000 separate assessments.

Water—While the water supply system is in charge of a commissioner the city engineer has practical control of all properties, including pumping station, standpipes, mains, and in fact all physical properties belonging to the department. Recently the commissioners have given the city engineer additional charge of the entire water-works office force, including reading and repairing of meters and the hearing of complaints as to excessive water bills. The actual work of the department, however, is under the supervision of the water-works engineer, who, of course, consults his chief in all matters of policy other than mere details of operation. Heretofore, the city of Montgomery by bond issue or otherwise has installed all of the standpipes, pumps, water mains, hydrants, etc., without assessment against the property, the same being paid for by general bond issues. Lately, however, a ruling has been adopted that if a water main or hydrants are to be put in in an undeveloped section, the property owner, or owners, make up the fund according to estimates of the engineer, and when this fund is in the hands of the city treasurer, then the city makes the installation.

It may be said in passing that all of the assessments are made up by the engineer's office. He even transfers the amount of the whole tax for each lot to the city treasurer's books. It is thought this information is all that the treasurer needs. If there is any question as to what items that total tax contains, the property owner is referred to the city engineer's office. The treasurer merely makes the collections.

Paving—A considerable mileage of brick paving was constructed by the city in the years 1894-1896. That is badly in need of maintenance now, and it is found difficult to maintain because of the absence from the market of the small bricks which went into the original pavement. The city has been doing a large amount of new work within the past two years, and almost all of it has been sheet asphalt work. Oil asphalt has replaced lake asphalt within the past year, the former being found as good a material and less expensive. In the last three years there have been about 300,000 sq.yd. of asphalt laid at an average cost of around $2.60 a yard. The sheet is 1½ in. thick with a binder of the same thickness, and is laid on a concrete base 5 in. thick of a 1:2½:5 mix. Sidewalks are generally of cement with a 2¾-in. base and a 1¼-in. top. This thickness varies only at driveways, where it is 6 in.

Granite curbing is laid invariably. When new paving work is done the asphalt is laid clear to the curbing. If there is already a gutter in, it is torn out and replaced by the same material of which the street is paved. This practice is followed because it has been found that where the paving and gutters were of asphalt and concrete, respectively, the gutter soon shreds at its joint with the asphalt. The result is holes between gutter and paving, with consequent danger to the subgrade through seepage. The best granite block obtainable is now being used for curbs and is set in place at a cost of 95c. a lin.ft. With the type of construction now in use it was found slightly difficult at first to convince contractors that a roller could operate flush with the face of the curb, but the contractors are now used to it and do a thorough job. There is a little asphalt squeezed up against the face of the curb which makes practically a watertight joint there.

Terra-cotta pipe is used altogether for sanitary sewer work. The sanitary system contains no pipe above 24-in. and that is now being installed to take care of the city's sanitary needs for many years to come. There are numerous outfalls into the Alabama River, and the sewage is given no treatment. Storm sewers are of terra cotta for the smaller sizes, and mostly concrete pipe with some monolithic sections for the main trunks. The city is practically 70 per cent serviced with sanitary sewers. There is a dry can collecting system for the unsewered section, which is mainly in outlying colored slum areas.

Were it not for the fact that about half of the surface water in Montgomery gravitates into one channel, the disposition of storm water might prove a problem. As it happens the city is cut in half by a natural waterway which has been straightened, cleaned and controlled by putting it into either pipe or box-section monolithic drain. The size of the section along this main outfall runs from 15 in. in diameter to a 10x20-ft. rectangular section. The city has to maintain the waterway from the city limits out five miles, where it reaches a larger stream. The need for extensive storm drain installations is seen when it is known that as intense a rainfall as 7.92 in. has been recorded in twelve hours. The yearly precipitation is about 50 in.

Montgomery is doing some research in connection with street paving which might be followed to advantage elsewhere. It has laid three parallel streets upon which the traffic conditions are almost identical with: (1) lake asphalt, (2) oil asphalt and (3) bitulithic. The two types of asphalt are built of a 1-in. binder with a 1½-in. top on a 5-in. concrete base, and the bitulithic is a 2-in. wearing surface on a 4-in. concrete base. It is intended that accurate maintenance figures on these three streets shall be kept and when replacement is necessary there will be accurate data upon which to design the new paving. The information will also serve as a guide to the construction of other streets.

Though traffic regulation is a police function in Montgomery, the city engineer lays out parking zones and advises as to the purchase of control signals. So far

Of the Small City

not much automatic signaling of traffic has been done, due first to the fact that the old part (and the most active business section now) is not laid out in the grid system, and secondly because of the width of the streets. Most of them are 100 ft. between property lines.

Though garbage collection is made by the commission of water supply and sanitation the physical equipment necessary is under the control of the city engineer. Garbage is dumped at the outskirts of the city and the maintenance of that dump is also an engineering function. Finally, other functions given Mr. Garrett which are seldom found are the indirect control of plumbing and lighting inspection, for both the city plumbing inspector and the city electrician report to the city engineer.

In conclusion let it be said that few city engineering offices are found where such complete accord exists among its principals. Mr. Garrett has a staff of young engineers who are efficient and active, but not too serious minded. The reason for such a well balanced staff is found when one spends several hours with Mr. Garrett himself.

Freezing in Wood-Stave Pipe at Everett, Wash.

EVERETT, Wash., is served by a 26½-mile continuous wood-stave pipe line which brings water to the city from the Sultan River. About 8 miles of this line is not covered and each year some ice collects in this exposed portion of the line during the winter time, but usually not enough to interfere seriously with the supply. Last winter an unusually hard freeze reduced diameter of the line to 8 in. and for a few days caused a water shortage. Conditions leading up to this situation and the results of the freeze are as follows:

In the Northwest the usual practice with regard to wood-stave pipe is to cover it with earth where possible. In some locations there is danger of freezing and there is always the fire danger and the deteriorating effect of sun and weather on its surface. In the construction of the Sultan River supply line the money available was not sufficient to cover the entire line and the only portions backfilled were those endangered by logging operations or some other special condition.

The line has now been in service nine years and the exposed portions have been checked and weathered so that considerable maintenance is required. With some repairs, the line can be expected to last for eight or nine years more if left where it is but it is not considered feasible to take the line down and move it, which would be required if it were to be put in a trench.

The excessive amount of freezing that occurred last winter is ascribed to a strong wind which blew continuously for several days of very cold weather. During this period it was noted that after a lining of ice 3 in. thick had formed inside of the pipe the ice thickness increased 1 in. every 24 hours. By the time the weather began to moderate the thickness had increased to 10 in. on the entire perimeter, leaving only an 8-in. opening in the center. About 8 miles of the 26, it is believed, was frozen to this extent. The thickness was determined by actual measurements through holes bored into the pipe and through the ice lining. Another check on the constriction of the diameter due to ice was made by calculating the diameter of a line that would deliver the quantity of flow measured at the outlet. This checked very closely with the 8-in. figure determined by direct measurements of the ice lining.

Nothing could be done to clear the ice from the line. However, a full pressure head was maintained to keep the line open if possible and provision was made for removing the ice at a sand box as rapidly as it thawed, so as to prevent clogging the line farther down. Fortunately an open sand box was located just below the section where the most serious freezing occurred. When the weather changed, the sun warmed the pipe line somewhat so that ice broke loose and was carried out into the sand box. Workmen were stationed there, ready to remove it, and no ice was allowed to continue on through the line. The velocity in the pipe was 3.5 ft. per second, which proved to be sufficient to carry through the fragments as they loosened and broke away from the inside of the pipe.

During this freeze there was considerable damage to the pipe of a sort which occurs to a lesser extent every winter, even when no ice gathers on the inside of the pipe, namely, the breaking off of the upper half of staves at the ends. In the construction of this pipe the joint between abutting ends of the staves is made tight by a metal key extending a few inches into each stave. As the pipe is under a head that ranges from 40 to 300 ft., the staves are saturated at all times, and when temperatures are low enough, particularly if the stave ends have a "slash grain," the portion of the staves above the key will be broken by the lifting action of the frost. This might not occur if circumferential rods were close enough. On this line the spacing of these rods is 5 in. and where joints occur between the bands there is always this danger in cold weather of spalling off stave ends for a few inches back from the joint. Operating experience with this line has led to the suggestion that the spacing should not exceed 2 in. where danger exists.

Repair of these spalled ends is made by placing a galvanized iron plate over each break and holding it in place by the use of two additional bands. These plates are ordinarily 6x8 in. in size and all four edges are sheared in the same direction so as to get a raw edge on the same side of the plate all around. A ball peen hammer is then used to tap these plates down into the wood, the raw edges giving a sufficient "bite" into the wood to make the covering watertight when held in place by the bands. In the worst section of this pipe line, 3,000 to 4,000 of these patches were put on within two miles.

For the foregoing information *Engineering News-Record* is indebted to Carl Klapp, superintendent of Everett Water Department.

Surface Treating Old Brick Pavement

During 1924 Rockford, Ill., resurfaced 13,513 sq.yd. of old vitrified brick pavement with a seal coat consisting of 0.57 gal. of 90 penetration asphalt. Coarse or torpedo sand 109 lb. to the square yard, was used as a covering. The asphalt was applied after thoroughly sweeping and flushing with a compressed air distributor under 15-lb. pressure. The cost was 7.5c. per square yard and of sweeping and flushing 50c. per 1,000 sq.yd. The work was done in August and is considered an experiment by Bernhard C. Harvey, city engineer.

Concrete Pump Pit Sunk as An Open Caisson

Walls Built on Steel Shoe—Forms Had Exceptionally Rigid Design—Water Jets Opening Out of Shoe Proved Ineffective

By B. Stanley Nelson
Chief Engineer, A. M. Lockett & Co., Ltd., New Orleans, La.

An interesting open caisson operation has recently been carried out by the contractor for the new city water-works station at Vicksburg, Miss. The water-works system there comprises: First, the low lift raw water pumps, which elevate the raw water from the Mississippi River into filter basins, where it is clarified and treated; and second, the high lift pumps, which take the filtered water from the storage basin, and deliver it to the city mains and standpipe.

At Vicksburg the Mississippi River has a maximum recorded range of water elevations from —6.5 to +55, a total range of 61.5 ft. To enable the raw-water pumps to take water at extreme low river, it is necessary to locate them in a deep pit. The ground line at the point where the pit was located was approximately El. +51, and the floor of the pit was fixed at El. +8, which would enable the pumps to operate successfully with the river at several feet lower than the present recorded low water. The top of the pit was made El. +58 to allow for unexpected elevations of high water. The floor of the pit was 5 ft. deep making the elevation of the extreme bottom of the caisson El. +3 and the extreme top El. +58, or a height of 55 ft., with an outside diameter of 19 ft. and an inside diameter of 17 ft., Fig. 1.

The site of the pit was carefully chosen by making exploratory borings with a 1-in. pipe under city water pressure of 115 lb. It was known that there was rock underlying this section, and the idea of the borings was to locate a suitable point where the bottom could be embedded in rock without excessive excavation. The point chosen had a level area of rock ample for the diameter of the pit at approximately El. +9 ft., which would allow the bottom of the pit to seal in the rock for about 5 ft. At the time the work was planned, it seemed likely from a study of the river records that the lower half of the pit would have to be sunk in wet material, for which reason it was decided to use the open caisson method of sinking.

Equipment and Forms—The equipment devised for this work comprised: (1) the caisson shoe; (2) collapsible circular steel forms, and (3) a gasoline-engine-driven derrick with half-yard clamshell bucket. The shoe was made as shown by Fig. 2 of ⅜-in. boiler plate, with all outside joints and rivet heads flush with the surface. It was shipped in two sections, which were bolted together on the job.

It was anticipated that there might be some trouble from the caisson going down crooked, and this contingency was to be met by means of water jets at the inside and outside bottom of the shoe. There were 36 of these jets, 18 inside and 18 outside, each jet consisting of a ¾-in. pipe nipple welded onto the shoe, and each three jets communicating with a single 1¼-in. pipe through a welded header. These 1¼-in. pipes were extended above the top of the shoe, and as each successive section of concrete was poured, as described later, they were extended by new sections equivalent to the length of each ring of concrete. It developed that these jets did not work as well as anticipated; they were really of no use on the job.

The collapsible forms are shown by Fig. 3. These were made of comparatively light sheet metal, securely stiffened by angles as shown. Both inner and outer forms were made in four sections each for convenience in handling. The inside forms had provision for adjusting the curvature, as it was feared the form might get out of round. It developed, however, that such provision was unnecessary, as the forms remained truly cylindrical, due to ample stiffening from the circular angles. The form sections had eye-bolts at proper locations for handling them by a derrick.

It was necessary to slot the holes in the bolting-up straps of the inner form sections, to compensate for the

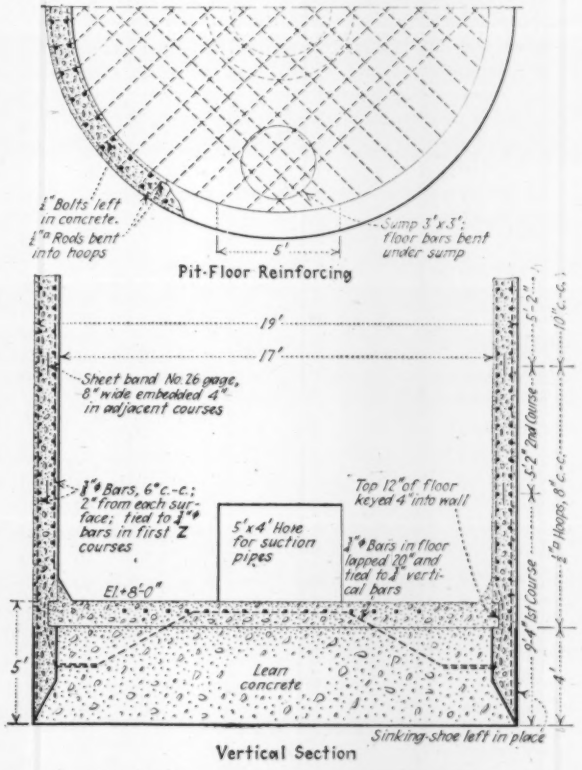

¾" Bolts left in concrete.
¾" ø Rods bent into hoops
5'
Sump 3'x3'; floor bars bent under sump
Pit-Floor Reinforcing
19'
17'
Sheet band No. 26 gage, 8" wide embedded 4" in adjacent courses
¾" ø Bars, 6" c.-c.; 2" from each surface; tied to ¾" ø bars in first 2' courses
5'x4' Hole for suction pipes
Top 12" of floor keyed 4" into wall
¾" ø Bars in floor lapped 20" and tied to ¾" vertical bars
El+8'-0"
5'
Lean concrete
Sinking-shoe left in place
Vertical Section

FIG. 1—PLAN AND SECTIONS OF CONCRETE PUMP PIT
Attention is directed to the reinforcing for the floor and the manner of locking the floor slab into the walls to resist upward pressure.

contraction of the radial bolts, to which the forms were bolted. These bolts were extremely useful in the construction as planned.

It will be noted that both the caisson shoe and the steel forms have a row of ¾-in. holes placed 6 deg. apart, both inside and outside for ½-in. bolts. These bolts served several purposes. When the shoe was filled with concrete, a set of bolts was embedded within the concrete with the ends projecting through the holes just mentioned. The inner form was then bolted onto the shoe by means of these bolts, and a second set of bolts was fastened at one end to the upper edge of the inside form. These projecting bolts were then used for fastening in the proper location the vertical and longitudinal reinforcing bars for the concrete, after which the outside sections of the form were bolted onto the

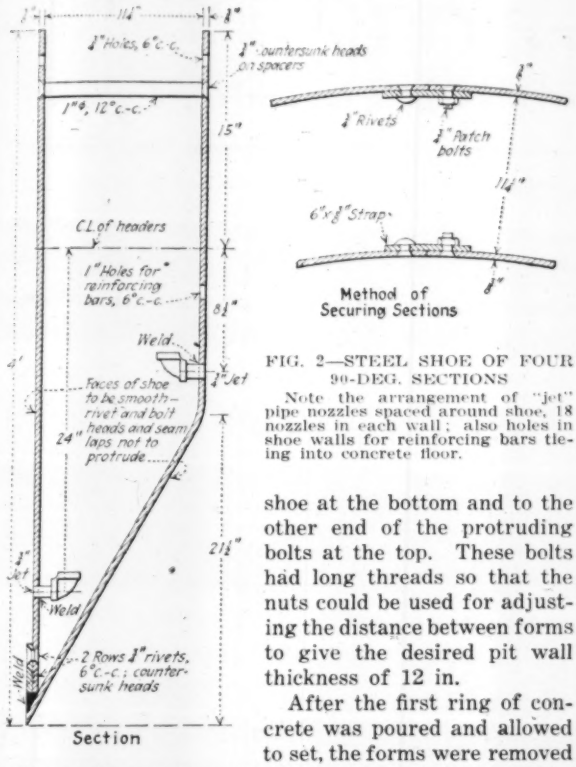


FIG. 2—STEEL SHOE OF FOUR 90-DEG. SECTIONS

Note the arrangement of "jet" pipe nozzles spaced around shoe, 18 nozzles in each wall; also holes in shoe walls for reinforcing bars tieing into concrete floor.

shoe at the bottom and to the other end of the protruding bolts at the top. These bolts had long threads so that the nuts could be used for adjusting the distance between forms to give the desired pit wall thickness of 12 in.

After the first ring of concrete was poured and allowed to set, the forms were removed and raised up another ring in height and bolted onto the bolts which had previously been at the top of the form. Each lift was 5 ft.

Construction Methods—The first step in the construction after the derrick was assembled was to dig a pit about 6 ft. deep of sufficient diameter to accommodate the shoe and allow working space on the outside. The shoe sections were then lowered into this pit and assembled, and filled with concrete, the idea being to avoid the necessity of wheeling the concrete to an excessive height, and yet keep the form assembling above ground.

The concrete used was a 1:2:4 mix of washed gravel ¾-in. and under, and sharp coarse sand. This was mixed purposely in a small mixer of one-half bag batch capacity, as it was thought that better mixing would result as well as better ramming by the use of small batches. The concrete was allowed to harden about three days before the forms were stripped and moved up.

Due to an unexpected low stage of the river, because of the drouth, the pit was sunk practically in the dry, for which reason it was found advantageous to dispense with the clamshell bucket, and excavate by hand, loading the material onto wooden skips, which were raised with the derrick and dumped on the spoil bank outside. It was found that the rate of lowering the pit was approximately one ring of 5 ft. per week.

Due to the extremely dry condition of the soil, when the pit was about 25 ft. down the caisson failed to sink further. The water jets were tried in an effort to lubricate the walls, but were unsatisfactory, and it was found necessary to probe with a pipe and water jet at intervals of several feet around the outside to soften the earth after which no trouble was experienced.

When the pit was 35 ft. down, an unexpected accident occurred. A leaky water main at one side softened the earth, which caved in and came up under the shoe into the excavation. The side thrust of this material rushing in at one bottom side with the unsupported earth pressure at the top opposite side, canted the caisson seriously. This trouble was overcome by taking several wraps around the outside of the caisson with a heavy steel cable, which was laid over the top and tied down to some heavy logs embedded in the river bank. This cable was tightened and fastened in such a way that as the caisson went down further it would cause a side pull, and it was found by excavating carefully under the shoe with an excess of excavation at the high side, that the caisson very readily straightened up again.

When the rock was reached at El. +9, some further trouble was experienced in sinking. The rock is a soft material, apparently a shale. It was impossible to

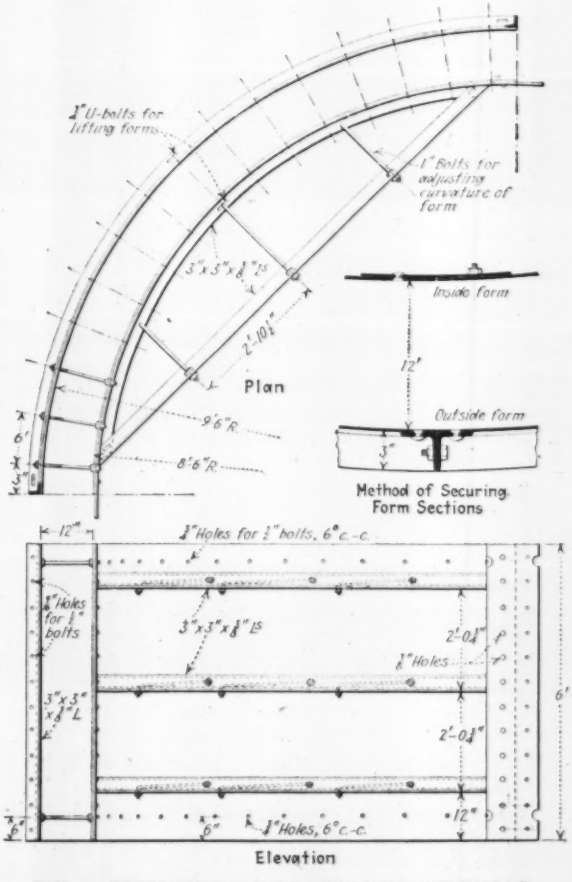


FIG. 3—FORMS FOR CONCRETE PUMP PIT WALLS

Note the angle stiffeners and the bolts for adjusting the curvature of the inside form.

jet into this, as was done earlier in the work, so the caisson was weighted down with a quantity of cast-iron pipe, valves and fittings, and gradually settled to its proper elevation, after which the heavy reinforced bottom was poured. This bottom is tied to the walls, first by the tapered inside section of the shoe, second, by the ¾-in. reinforcing bars projecting from the inside of the shoe, and third, by a substantial keylock, which is made into the walls of the pit just above the top of the shoe.

The total time occupied in sinking the pit from the time the shoe was assembled to the completion of pouring the bottom was approximately 64 days.

From Job and Office

Hints That Cut Cost and Time For the Contractor and the Engineer

Rapid Winter Concreting on St. Louis Municipal Market

IN THE construction of the new municipal market and garage at St. Louis, Mo., the limited storage space at the site necessitated a large bin capacity to provide for rapid progress in the mixing and placing of 12,500 cu.yd. of concrete, containing 800 tons of reinforcing steel. A bin capacity sufficient for three or four hours' continuous pouring was considered necessary to take care of possible delay in the delivery of materials by trucks. This reinforced-concrete building is 195½x270 ft. in plan, with a column spacing of 28½x30½ ft. It is four stories high, with steel roof trusses carried by steel wall columns in the fourth story.

The number of construction joints was limited by the specifications to one longitudinal and three transverse joints in each floor. A day's pour averaged 325 to 340 cu.yd., and there were 2,600 to 2,700 cu.yd. per floor, exclusive of the columns, as a construction joint was allowed at the base of the column capitals. This meant, in an eight-hour working day, the pouring of at least 40 cu.yd. per hour. The time of mixing in the mixer was specified as two minutes, which made the total time per batch approximately 2 min. 20 sec. It was therefore necessary to use a battery of two 1-yd. mixers in order to keep up with the program. The mixers were placed at basement floor level on one side of the building, with the material bins overhead and discharging directly into the mixers. This arrangement made it necessary to use non-tilting mixers of the drum type in order to conserve in height, as the material bins were supplied by the trucks from a driveway on top. The mixers discharged directly into a single bucket with a quick shift arrangement. The latter device, with a sliding frame for a take-off at any height, made only one tower necessary. All the concrete was spouted directly into the forms.

This concreting plant, with steam hoist, mixers, tower and material bins was erected in the street. The bins had a capacity of two car-loads of sand and four car-loads of gravel; cement was stored in a house of one-car-load capacity, adjacent to the tower and mixers. All concrete was a 1:2:4 mix, with 8 per cent of hydrated lime; for the finishing surface of the floor a 1:2 mix with traprock was used, deposited to form an integral part of the slab. Steel forms were used for the columns and wood forms for the floors, the latter being used twice as a rule. Steel chairs supported the reinforcing rods in the floor forms. Precast concrete roof slabs were carried to the fourth floor in motor trucks running on the ramps which serve the garage. Forms and falsework were left in place for a minimum of 18 days, with 12 days re-shoring.

The total amount of excavation, 29,000 cu.yd., was accomplished with two steam shovels (both in operation only 60 per cent of the time) in fifty days, which was an average of 580 cu.yd. for a nine-hour working day. In concreting, an average of about three weeks

CANVAS ENCLOSURE FOR WINTER CONCRETING
Wooden trusses support canvas roofing.

was necessary for the construction of each floor. This required the building of over 50,000 sq.ft. of forms and the pouring of about 3,000 cu.yd. of concrete. The maximum pour for a single day was 381 cu.yd. The erection of 250 tons of structural steel in the fourth story and roof was accomplished in eight working days. A construction force averaging 180 men was maintained for the rapid progress of the work.

As the building had to be constructed during the winter of 1924-1925, a method of concrete protection was devised, consisting of canvas roofing stretched over wooden trusses supported at the column and valleyed to drain off all water. In this way the area of one entire floor was enclosed for concreting. The canvas was bellied out over the spandrel beams to allow free

(Continued on p. 1082)

Two Useful Cost Report Forms for Contractors

TWO forms for reporting and recording costs, which have been worked out particularly for use in sewer and water-works construction, are shown to small scale in the accompanying illustrations. The "equipment record" is a consolidated form made up from a number of others which have been published and, except perhaps in the consolidation, presents nothing original. One sheet, it will be noted, contains the record of a machine during its life. The illustration shows only

FORM 1—FORM FOR LIFE RECORD OF A UNIT OF EQUIPMENT
Original sheet 8½x11 in.

the face of the form. Its reverse is ruled to record any rental income the machine may bring. This form being a permanent record is printed on good quality cardboard. Form 2 is a foreman's daily report form and is notable for embodying costs and receipts on the same form thereby definitely showing the actual profits or loss of each foreman on the daily report of foremen. This sheet is made up from the daily time distribution sheets handed in by the foremen.

These forms have been worked out and found successful in use by J. Ray McAuliffe, general superintendent for J. D. Hanley, general contractor, Portland, Ore.

J. D. HANLEY---GENERAL CONTRACTOR

709 GASCO BUILDING, PORTLAND, ORE. — FIRST AND CEDAR STS., KELSO, WASH.

EQUIPMENT RECORD

ARTICLE _____ NUMBER _____

DESCRIPTION _____

MFG'S NAME _____

MFG'S SIZE _____ MFG'S TYPE _____ MFG'S NO. _____

PURCHASED FROM _____ DATE _____

PURCHASE PRICE OF ARTICLE _____

ACCESSORIES _____

" _____

" _____

" _____

" _____

TOTAL INITIAL COST _____

FREIGHT $ _____ HAUL _____ ERECTION $ _____

TOTAL COST IN PLACE _____

ESTIMATED LIFE _____ YRS. ANNUAL DEPRECIATION $ _____ SALVAGE VALUE $ _____

JOB RENTAL RATE $ _____ PER MONTH. WEIGHT _____ LBS.

CONDITION OF PURCHASE _____

REMARKS _____

DEPRECIATION AND INVENTORY VALUE

AGE	FIRST VALUE	DEPRECIATION	INVENTORY	AGE	FIRST VALUE	DEPRECIATION	INVENTORY
1 Year				6 Years			
2 Years				7 Years			
3 Years				8 Years			
4 Years				9 Years			
5 Years				10 Years			

REPAIR RECORD

DATE	DESCRIPTION OF REPAIRS	COST

FORM 2—FOREMAN'S DAILY REPORT FOR SEWER CONSTRUCTION
Original form 11x8½ in. Waterworks form closely similar

FORM JDH-109

J. D. HANLEY
GENERAL CONTRACTOR
709 GASCO BLDG., PORTLAND, ORE.
301 MASONIC BLDG., KELSO, WASH.

JOB _____

LOCATION _____

DATE _____ 192__

DAILY REPORT OF FOREMAN

COST OF LABOR: MEN	HRS.	RATE	AMOUNT	HAND EXCAVAT'N	BACKFILL	PIPE LAYING	PIPE HAUL	LUMBER HAUL	CONCRETE MATERIAL HAUL	LAGGING	FLOORING	CONCRETE BASE	MANHOLES	FLUSH TANKS	SUMP	PUMPING	EXTRA WORK
TOTALS TODAY																	
BROUGHT FOR'D																	
TOTALS TO DATE																	

(Sewer Construction Form)

(Note: Use reverse side for calculations & Unit costs, progress etc)

WORK DONE

ITEM	STA TO STA	LIN. FT.	QUANTITY	UNIT	BID	AMOUNT	REMARKS
							Todays Receipts $
					Contract Prices		Labor $
							Pipe $
							Gravel $
							Cement $
							Lumber $
							Etc $
							$
							Todays Expense $
							S or D
							Forward S or D
							S or D
			TOTAL TODAY				
			BROUGHT FORWARD				
			TOTAL TO DATE				Note: This figure represents total estimate to date

Actual Labor and Material Costs

circulation of air heated by salamanders placed on the floor below. The concrete materials were also heated before being deposited in the forms. Live steam was discharged into the sand, gravel and water before entering the mixer. In freezing weather, the concrete had an average temperature of about 60 deg. F. when leaving the mixer and 50 deg. when deposited in the forms, having lost about 10 deg. in an average travel of 250 ft. in the tower and chutes. No concrete was poured in weather under 20 deg.

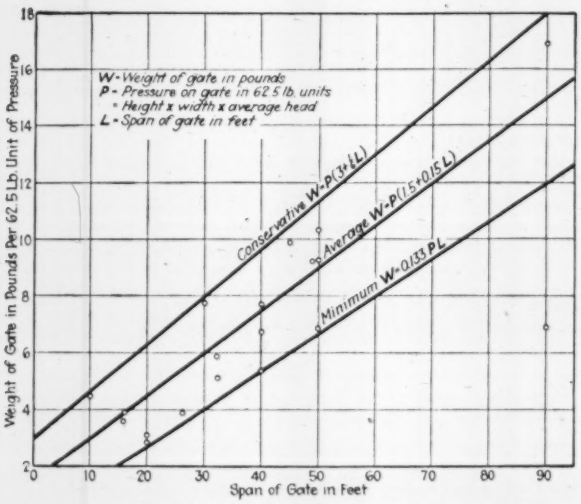
This municipal structure was designed in the bridge and building division of the Board of Public Service; L. R. Bowen, chief engineer; E. E. Christopher, architect; W. R. Crecelius, civil engineer; and F. N. Hatch, engineer in charge of construction. The general contractor was the Fruin-Colnon Construction Co., St. Louis, with C. F. Newes as superintendent and James D. Tancil as resident engineer.

Weight of Stoney Sluice Gates

By E. W. Lane
Detroit, Mich.

IN THE study now being made of the improvement of the St. Lawrence River for navigation and power development, the question of the cost of Stoney sluice gates came up. A search of engineering literature revealed considerable data on the weights of gates of different dimensions. This was analyzed on the accompanying diagram.

This diagram gives the weight in pounds per 62.5 lb.



ESTIMATING CHART FOR STONEY GATE WEIGHTS

unit of pressure on the gate, for gates of various spans. To find the weight of a gate, take the product of its width, its height, the average head, or head at the center of the gate, and the value shown from the curve for a gate of the given span. Three curves are shown, with a corresponding formula for weight of gate. The proper curve to use will depend on the degree of conservatism desired in the estimate. In some cases it was not possible to determine from the literature if the weights given included the guides and rollers and values near the conservative curve probably include them. They do not include the weight of towers, hoists or counterweights.

These curves can also be used in estimating the weights of sliding gates built of structural shapes.

From Job and Office

Hints that Cut Cost and Time

Hydraulic Machine Bends Reinforcing Steel on Large Sewer Job

IN BUILDING the Flatbush relief outfall sewer in Brooklyn, N. Y., a $3,250,000 project, consisting of something over three miles of reinforced-concrete sewer, manual methods for bending the reinforcing steel were found to be very inefficient and costly. The outgrowth of the condition was that the contractor

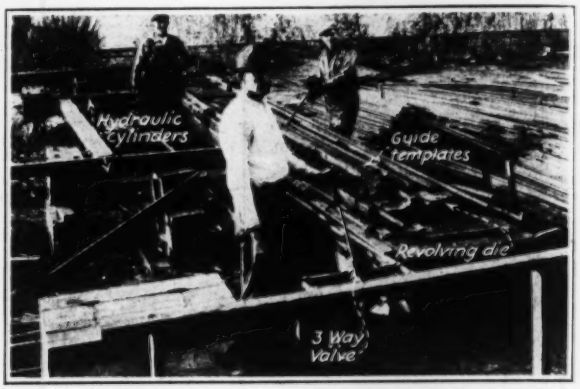


FIG. 1—HYDRAULIC MACHINE FOR BENDING REINFORCING STEEL

designed and built a machine in which hydraulic cylinder mechanisms (Figs. 1 and 2) bend the reinforcing bars to the proper shapes. Thus, all bending is done at a central bending plant and the steel is transported by truck to the construction operations.

The machine consists of a table equipped with a system of guide templets and on either end a set of revolving dies. Some of the templets are fixed and some are loose as shown in Fig. 2. The latter are moved by hydraulic pistons, thus bending the steel to the required shape. All steel is bent into either double or single truss bars with hooked ends. The latter are formed by the revolving dies. The first operation consists in placing the truss in the bar by means of the cylinder piston rods. The cylinders are hoist cylinders taken from old motor trucks. The two cylinders move together and are operated by a three-way valve. This three-way valve permits pressure to be put on either side of the piston, thus moving it either forward or backward. The guide templets at either end are fastened to a sliding plate which can move in either direction, thus taking care of the shortening of the bar resulting when it is trussed. This sliding plate is plainly shown in Fig. 1.

After the truss is placed in the bar the ends of the bar are fastened in the revolving dies. These also operate together, the hydraulic pressure being controlled by means of another three-way valve directly across the table from the one operating the guide templet pistons. As shown in Fig. 2, the revolving dies are operated by a separate pair of hydraulic pistons placed longitudinally beneath the table. A large gear wheel is mounted horizontally on the vertical shaft of

From Job and Office

For Contractor and Engineer

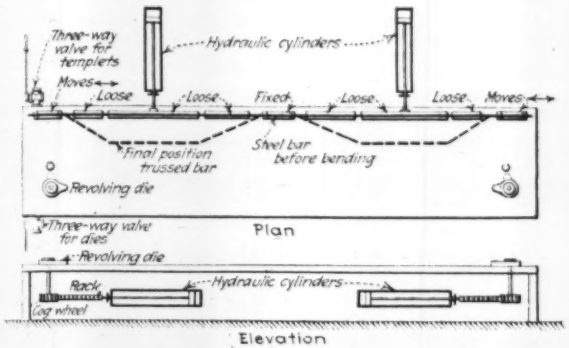


FIG. 2—HYDRAULIC BENDING MACHINE WITH DETAILS OF OPERATING MECHANISM

the revolving die. This gear engages a rack attached to the hydraulic cylinder piston.

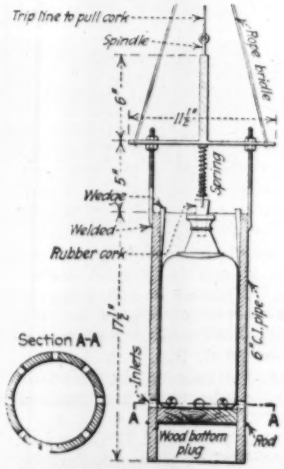
Pressure is supplied by a triplex pump getting its water supply from a temporary steel tank. A relief valve is placed in the high pressure line of the pump and set at 120 lb., which is the maximum pressure used on the machine. Oil is placed in the water to lubricate the cylinder parts. As much as 25 tons of double truss steel has been bent and hooked in 8 hr. with this machine.

The sewer is being constructed by the Montrose Contracting Co., under the direction of the Bureau of Sewers, Brooklyn, Arthur J. Griffin, chief engineer.

Sounding Bottle for Taking Samples of Water in Reservoirs

A HOME-MADE device designed to simplify taking samples of reservoir water at varying distances below the surface has been developed by the water department of Everett, Wash. Details are shown in the accompanying illustration.

The device consists of a one-gallon bottle fitted into a section of 6-in. cast-iron pipe and equipped with a rigid frame at the top supporting a rod and a spring. The rod is guided by a 6-in. length of tubing so that when the rod is withdrawn and then released the spring forces it back into its previous position. By this means a cork attached to the end of the rod can be withdrawn and then replaced by pulling and releasing a trip line. The method of operation is simply to lower the weighted bottle to the desired depth, as indicated by marks on the sounding line, and then to draw the cork by means of the trip line. When bubbles no longer come up the trip line is released and the spring returns the cork to its place.



HOME-MADE RESERVOIR WATER SAMPLER

It is to be noted that this spring should be strong enough to thrust the cork firmly into place and hold it there, but not so strong that the pull on the trip line required to compress it will raise the device while its effective weight is decreased by the buoyancy of the empty bottle. With a spring of the right tension a quick jerk will release the cork without materially disturbing the position of the bottle.

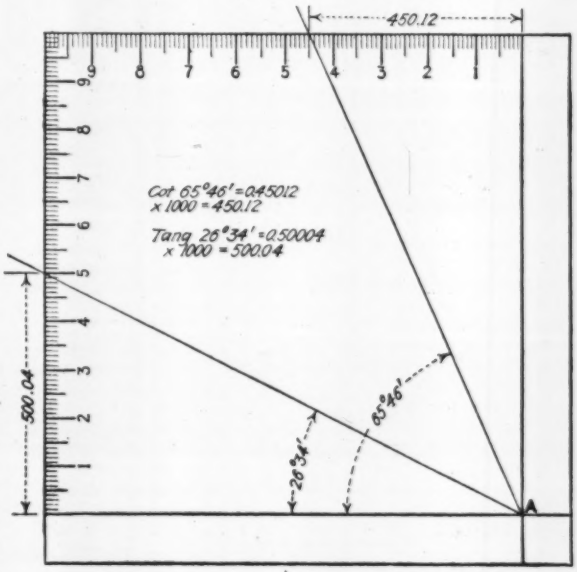
For the foregoing information *Engineering News-Record* is indebted to Carl F. Klapp, superintendent, water department, city of Seattle.

Protractor for Laying Out Angles by the Tangent Method

By William C. Reisner
Engineering Department, City of Glendale, Calif.

THE instrument herein described can be made easily from the celluloid used for windows in automobile side curtains. The manner in which it is made is as follows:

First lay out a perfect square just larger than the



PROTRACTOR FOR ANGLE MEASUREMENTS

scale to be used. Lay out any scale to represent 1,000 ft. on two adjoining edges, then lay out the lines through the zero points of the scale and parallel to the scaled sides. When this is done the instrument is then ready for use.

Place the intersection *A* over the point where the angle is to be laid off and one of the lines intersecting at *A* along the course from which the angle is to be laid off. Lay off tangent (times 1,000) of the angle on the scale from the initial course and connect with *A*. If the angle is greater than 45 deg. use the cotangent (times 1,000) on the scale parallel with the initial course. A scale 10 in. long is very convenient and accurate.

It will be seen that this same instrument can be used to locate co-ordinated points.

Letters to the Editor

This department aims to be a forum for the discussion of the views of engineers and contractors. The range of interest should be as wide as possible. Contributors are, therefore, asked to make their letters short.

Not So Favorable to Admixtures in Concrete

Sir—The statements made in Mr. Conners' article "Concrete Improved by the Use of Diatomaceous Earth," in the Dec. 17 issue of *Engineering News-Record*, are so much at variance with the writer's experience with powdered admixtures and conclusions based thereon, that it may be of some value to compare such widely divergent viewpoints.

The results of investigations of the use of powdered admixtures by the Bureau of Standards and by the Structural Materials Research Laboratory have been borne out by the experience we have had in the placing of concrete in reinforced-concrete railroad bridges, i.e., that the compressive strength of concrete is dependent on the ratio of the volume of water to the volume of cement in the mixtures, regardless of proportions with or without powdered admixtures.

Using proportions of aggregates giving maximum workability, we have obtained a quality of concrete the uniformity of which is substantiated by the excellent working conditions during placement, the appearance of the completed work and the results of one thousand 6x12-in. test cylinders. A cylinder was taken from each 40 cu.yd. of concrete placed.

The use of powdered admixtures, to obtain the same workability, in all cases required an additional quantity of water, which reduced our compressive strengths accordingly.

More careful proportioning of concrete mixtures with graded aggregates will insure satisfactory workability.

If arbitrary proportions are specified and additional workability is required, it will usually be more economically accomplished by using additional portland cement. The additional cement will permit the maintenance of the proper water-cement ratio and insure a more durable and dense concrete.

The reference to increased yield is not peculiar to diatomaceous earth alone as the volume of concrete is increased by the mass of any powdered admixture used together with the necessary additional volume of water required to make the admixture plastic.

The percentage of the increased volume of concrete obtained depends on the density of the arbitrary aggregate proportions.

T. P. WATSON,
Assistant Engineer, Chief Engineer's Dept.,
Pittsburgh, Pa., Pennsylvania R.R.
Dec. 22, 1925.

The Bureau of Standards and Zoning

Sir—I have read the editorial note in your issue of Nov. 26, 1925, p. 861, criticizing the Bureau of Standards in the Department of Commerce for preparing and distributing information and statistical material on the subject of building zoning and city planning. The ground for your criticism seems to be that it is a federal interference with state duties. You seem to forget that the federal government has been of the greatest aid in preparing and distributing information and statistics in many fields which are as entirely within state jurisdiction as zoning and city planning. The government has spent millions of dollars to aid agriculture and mining. You will look in vain in the United States Constitution for any grant of jurisdiction from the states to the federal government on these subjects. All of the people of this country have been grateful to the federal government for the information sent out on these subjects. The present Secretary of Commerce apparently thinks that it is as proper to advise city dwellers as country dwellers on the prevention of waste. Modern zoning and city planning are more and more preventing enormous waste in cities. Now that so large a fraction of our population is in cities, it is difficult to see why the government printing office should not distribute information to help the cities as well as the agricultural or mining districts. Secretary Hoover is certainly entitled to the greatest praise for entering this long neglected field. The Bureau of Standards has not suggested the passage of any federal law on these subjects. There is no interference whatever with the free legislation on these subjects by states.

E. M. BASSETT.
New York City,
Dec. 14, 1925.

[Although the note mentioned by Mr. Bassett did raise a question as to the wisdom of federal studies of zoning it was designed more pointedly to urge that such a line of investigation is not a proper function of the Bureau of Standards, which we understand to have been created for technical rather than political research.—EDITOR.

Parabolic Sections for Banked Curves

Sir—Owing to preoccupations of business I have only just had an opportunity of reading Mr. Myers' letter which appeared in *Engineering News-Record*, Oct. 29, 1925, p. 728.

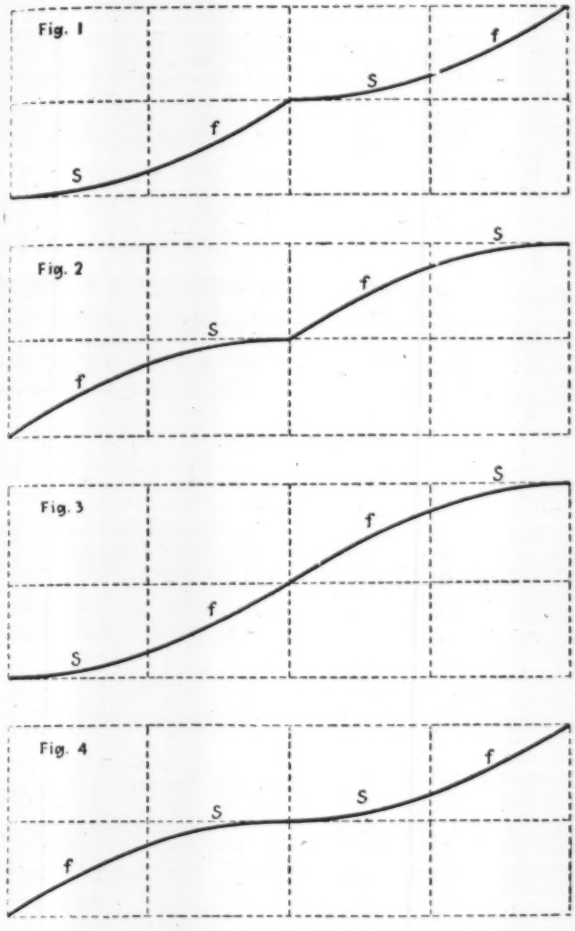
Considered as a general statement of the theoretical considerations that might be expected to influence the design of a cross-section of a roadway on a curve, there is much in Mr. Myers' letter which can be agreed upon at once. But there the matter ends, for it can hardly be contended that the arguments given in the letter can be translated into practical facts on the surface of a highway. In normal civilized communities there is a natural arrangement for the flow of traffic along the road, and this arrangement is independent of any local rules as to the use of the right side or the left side of the road by advancing vehicles. It is the natural arrangement that fast traffic will pass outside slow traffic, and whatever other rules may exist for the regulating of traffic this rule always remains. In discussing any question relating to the practical arrangement of a working road this must be taken as axiomatic for it can hardly be conceived that traffic, particularly in towns, can be regulated in any other manner having regard to the conditions under which roads are used. The stopping vehicle must obviously pull up and discharge its passengers or goods at the side of the road, and, therefore, slow-moving vehicles must pass nearer to the center of the road and fast-moving ones outside this again.

The discussion in the original article deals with country highways and these, of course, on through roads must considerably exceed in mileage the portions of the roads passing through towns. But the arrangement of the stream of traffic must surely be settled on the country highway as on the town street, because the same conditions apply as to stopping and turning out along branch roads, even though congestion of vehicles is not so great and the proportion of stopping and slow-moving traffic much less. In any case if a definite stream is fixed by town traffic it must obviously be extended along the country highways, for any rearrangement would mean the crossing of slow traffic over the fast at some points.

The accompanying figures 1 to 4 are copies of the sketch of corresponding numbers in Mr. Myers' letter of Oct. 3 in which the cross-sections have been relettered. Instead of the *a*, *b*, *c*, *d*, which he uses, the various divisions of the cross-section are marked *s* and *f* to indicate the lines of slow and fast moving traffic respectively. The only sections which are symmetrical about the center line of the roadway in regard to arrangement of traffic are combinations 3 and 4, so that in the cases shown in Figs. 1 and 2 different rules of the road would have to be applied to the streams of traffic in the two directions, which is obviously an unworkable proposal. The section shown in Fig. 4 would be quite usable provided fast traffic moved at the sides of the road and slow in the center, an arrangement the inverse of the normal, as well as having disadvantages to which Mr. Myers refers. The section shown in Fig. 3 is the only one in which a variable cant proportionate to the speed of the traffic is given and at the same time the normal arrangement of flow of traffic is preserved.

The argument that because the quantity of fast-moving traffic is several times greater than that of slow, the former are to be put on the sides of the road so as to leave the center of the road as clear as possible, may be quite sound

from the abstract point of view but could hardly be applied in practice. There are few drivers of vehicles of any sort who do not, when the road is clear, drive in the center of it. To expect fast-moving traffic to take to the side amounts to



FIGS. 1-4—COMBINATIONS OF TWO-WAY PAVEMENTS

expecting every driver to make complete inversion of his ideas of driving.

It is true that we do not expect two vehicles coming in the same direction to pass on the curve but that expectation will not prevent the fact from happening. One has only to consider such a case as that of a fast car passing some slow vehicle such as a farm cart or a heavy tractor train on a long curve to see the futility of assuming that vehicles will always follow in Indian file in such a case.

In conclusion, while Mr. Myers' contentions are interesting as theoretical abstractions, the practical conditions which obtain by reason of the fact that roads are used by all sorts of people and all conditions of vehicles make the greater part of these considerations of academic interest only and bring us back to the profile shown in Fig. 3 as being the only parabolic profile that is suitable for use on banked highways.

E. G. WALKER.

Westminster, England.
Nov. 27, 1925.

Spiral Curves in North Carolina

Sir—I wrote an article for *Engineering News-Record*, which appeared on July 24, 1924, p. 132, entitled "Spiral Curves Cut Road Costs in North Carolina," in which I failed to acknowledge my indebtedness to O. B. Bestor and George F. Syme for their constructive criticism in the design of this spiral. It was purely an oversight, and I trust you will publish this letter as an apology for my negligence.

CHARLES M. PRITCHETT,
Locating Engineer, North Carolina State Highway Commission.

Asheville, N. C.,
October 26, 1925.

River Channels and Flood Flow

Sir—It seems to me that in reviewing the book by George W. Pickels on Drainage and Flood Control, in your issue of Dec. 17, Arthur E. Morgan has gone considerably out of his way to find fault when he states: "On page 55 there is a curious recrudescence of fundamentalism in the statement, 'Streams have carved out their channels and flood plains to provide for their flood flows, and any encroachment by man in the way of levees, bridges, or buildings is naturally followed by more frequent flooding and greater flood heights, opinions to the contrary notwithstanding.' In fact, the size of stream channels has no necessary relation whatever to the flood flow. The Hudson River below Albany has dug for itself a channel through the hills large enough to hold a hundred times the maximum flow, while the Coldwater River in Mississippi and the Little River through the flat land of southeastern Missouri, in their unimproved conditions, would carry only about 10 per cent of the maximum flood flow. Of course any encroachment upon the low lands over which flood water passes will cause a raising of the flood stage."

The statement made by Mr. Pickels is absolutely correct. It seems out of place for Mr. Morgan to apply the opprobrious epithet "fundamentalism" to what really is fundamentally correct.

Mr. Morgan's statement that "the Hudson River below Albany has dug for itself a channel through the hills large enough to hold a hundred times the maximum flow" is erroneous. The fact is that the Hudson River is flowing in a sunken valley. There can be no doubt that in the geologic age when the Hudson River was reducing its base level and carving out its channel, it did so to the extent that was necessary to accommodate the average low-water runoff and that the flood plain, now submerged, held the flood waters.

Even as conditions are now, any restrictions placed in the channel of the Hudson River will have the effect of causing a rise, however small, in the surface above, which is in accord with the last sentence of the quoted portion of Mr. Morgan's review.

R. REIMANN,
Assistant Engineer in Charge of Surveys, Baltimore & Ohio R.R.

Baltimore, Md.
Dec. 21, 1925.

Temporary Hinges in Arch Ribs

Sir—J. F. Brett's statement by letter in *Engineering News-Record*, Nov. 5, 1925, p. 766, that the European method of designing concrete arch ribs with temporary hinges is not sufficiently understood or appreciated on this side of the Atlantic, moves me to say that the method has been in use by me for many years on all my arches that are not small enough to be poured in one day to have the concrete at the spring plastic until the full weight of the ring is on the centering.

Theoretically the hinge provision should be continued until all dead-load is on the arch, or rather such portion of it as would bring the line of pressure to the point at which the hinge is placed, but this is not always practicable, and the deformation of the hardened ring is trifling.

The Walnut Lane bridge, Philadelphia, 240 ft. in main arch, designed in 1906 and described in *Transactions*, Am. Soc. C. E., 1909, had six temporary hinges—three each side—consisting of small concrete posts at the neutral axis, embedded, of course, when the closing keys were poured. They demonstrated their utility by cracking on the upper side in the course of bending as the falsework deformed under the increasing weight of the ring.

A combined steel-plate and concrete arch of 262 ft. span, built in 1907, described in *Proceedings*, National Association of Cement Users (American Concrete Institute), 1908, was erected on temporary hinges of zinc, and then was made fixed with steel wedges at a time of mean atmospheric temperature, and at a stage of load that would, by the elastic theory cause the line of pressure in the fixed arch to pass through the middle of the ring where the hinge was placed.

In a multiple arch bridge now under construction without falsework (forms supported by steel ribs embedded) and giving unbalanced erection thrusts against the piers, I am placing the temporary hinges—heels of steel ribs and small concrete posts—at the lower third point of the rings to both reduce the thrusts and, by initial stress, keep the working line of pressure in the ring preferably low.

HENRY H. QUIMBY,
Consulting Engineer.

Philadelphia, Pa.,
Dec. 19, 1925.

News of the Week

CURRENT EVENTS IN THE CIVIL ENGINEERING AND CONTRACTING FIELDS

WASHINGTON NOTES

AN EFFORT to take the Tennessee River out from under the Waterpower Act is gaining momentum. Legislative steps already are being taken to that end.

Senator McKellar, of Tennessee, has accused the Corps of Engineers of improprieties in making public information as to the survey of the Tennessee river before submitting it to Congress. He also criticised the Chief of Engineers, on the floor of the Senate, for his refusal to postpone a local hearing at Chattanooga on Tennessee river applications.

Following Senator McKellar's comment in the Senate, Representative Dempsey in the House defended the action of the Corps of Engineers and put into Congressional Record Gen. Taylor's reply to Senator McKellar, which pointed out that the Chattanooga hearing was in accordance with the provisions of the Waterpower Act. "Refusal to hold hearings," Gen. Taylor said, "when the full requirements of the law have been met, as they were in

(*Concluded on p.* 1088)

Engineering Fifty Years Ago

From *Engineering News*, December, 1875

To Our Readers

WE HOPE our friends will appreciate the international character of our paper—that we have more subscribers in Pennsylvania than in Illinois, in Canada than in the state of New York, in California than in Maine, in England and South America than in Vermont and New Hampshire together,—and thus feel that we do not belong to any particular section of country, but that the interest of every part of the continent is equally entitled to our consideration. We hope thus to elicit a correspondence and interchange of opinion, that will make us more familiar with the great resources of this country, and of the various methods of developing them.

Hearing on San Francisco Bay Bridge Project

Thirteen Projects for Bridges and Tunnels or a Combination of Both Presented

A public hearing on revision of War Department requirements as to a bridge across San Francisco Bay was held Dec. 11, 12 and 18 at San Francisco by Maj. John W. N. Schultz. It was the third held by the War Department within four years on the general subject of Bay bridges at San Francisco, and was called at the request of the San Francisco Board of Supervisors, who have pending several applications for bridge franchises that would conflict with the War Department's present rulings (summarized in *Engineering News-Record* July 30, p. 178). San Francisco County, being on "the left bank," has authority under the state law for granting the franchise which must precede a federal permit.

Although the hearing was called primarily to receive information "bearing on the necessity, advisability and possibility" of modifying the War Department's policy with regard to any Bay bridges at San Francisco, the

THIRTEEN SAN FRANCISCO BAY BRIDGE AND TUNNEL PROJECTS PRESENTED

Proponents	Engineers	Type of Structure Proposed	Location	Channel Crossing (135 Ft. Above Water Level Unless Otherwise Noted)	Other Main Spans
"Victory Tubes" James M. Oliver	Davies and Modjeski	Combination bridge and tube (one R.R.; two vehicular tubes)	Foot of Folsom St., San Francisco to S. P. Oakland Mole	3,000-ft. channel 50 ft. clear depth	High bridge, including one 1,000-ft. span
Bay Cities Bridge Co. (A. J. Hanford)	John G. Little	High level steel trusses	Foot of 16th St., S. F., to foot of Pacific Ave., Alameda	One 1,000-ft. span Two 590-ft. spans	Forty-eight 250-ft. steel trusses
A. H. Bergstrom	John B. Leonard	High level steel trusses, double-deck	Hunter's Point to Webster St., Alameda	Six 510-ft. spans	200-ft. clear spans, towers 50 ft. wide
"Sunshine Transbay Boulevard Bridge" J. E. Birmingham	Joseph B. Strauss	Steel trusses, Y on East end serves both Oakland and Alameda	7th and Mission Sts., via elevated to Hunter's Point, thence to Alameda near estuary	1,050-ft. cantilever span near S. F. side; 900-ft. cantilever over estuary	20,400 ft. in deck spans
Allan C. Rush	Allan C. Rush	"Interlocking cable suspension bridge"	Foot of Mission St. to Key Route Mole via Goat Island	2,400-ft. spans 200 ft. above water level	.....
.....	R. S. Chew	Three-truss double-deck cantilever	Telegraph Hill to Goat Island, thence parallel to Key Mole	One 1,850-ft. span 180 ft. above water, similar to Quebec Bridge	Two 1,700-ft.; one 1,380-ft.; one 635-ft.; eight 200-ft.
.....	Chas. E. Fowler	Double-deck cantilever (suspension alternative)	Telegraph Hill to Goat Island, thence parallel to Key Mole	Three 2,000-ft. cantilever spans, piers 250 ft. wide, 235-ft. clearance	Two 1,350-ft. approaches
.....	Ed. A. Bertram	High level steel trusses	Foot of Folsom St., San Francisco, to S. P. Oakland Mole	Six 1,500-ft. spans Two 1,000-ft. spans	Fourteen 715-ft. spans
O. A. Ellis	O. A. Ellis	Two tunnels	4th and Harrison Sts., S. F., to 7th and Cedar Sts., Oakland	200 ft. or 300 ft. below water level	(Depth to be according to foundation conditions)
Construction Co. of North America	A. J. Cleary and J. J. Walsh	High level steel trusses, double-deck	Rincon Hill, S. F., to S. P. Oakland Mole	1,050-ft. cantilever span. Three 600-ft. spans; one 550-ft. span	Chiefly 200-ft. clear spans with 50-ft. towers
New York and San Francisco Development Co.	W. C. Earle	High level steel spans	Hunter's Point to Bay Farm Is., Alameda	Four 750-ft. clear cantilever spans	Forty-three 250-ft. spans
"Air Line High Bridge"	Roy C. Hackley	High level cantilever	Pier 26 on S. F. side to S. P. Oakland Mole	Two 1,800-ft. cantilever spans; 1,000-ft. span over estuary	Chiefly 400 to 700-ft. spans
.....	Couchot and Rosenwald	Tunnels	Foot of Harrison St., S. F., to S. P. Oakland Mole	50-ft. clear depth for a length of 1¼ miles	.....

merits of individual projects for bridges, tunnels or combinations—13 in all—were presented and discussed. Speakers representing Berkeley, Alameda and San Francisco as well as state organizations and cities elsewhere in the state, made presentations tending to show the urgent need for a continuous roadway across the Bay and urged that the construction of some crossing be approved. Oakland registered a dissenting expression as to this need, fearing that a bridge might interfere with future water traffic. The Navy Department was of the opinion that several of the proposed crossings would interfere with the proposed naval base, would restrict tidal flow or in some way interfere with naval operations, and pointed out that of the crossings thus far proposed "there was less objection" to a high bridge from Telegraph Hill than to others. In the following are given some of the outstanding points in the discussions.

In opening the session Maj. Schultz mentioned that the War Department would consider whether any proposed structure was "an *unreasonable* obstruction to navigation." Referring to this several speakers scored the Navy Department for objecting to any and all obstructions regardless of the urgent need for a bridge; and also for imposing onerous channel requirements that would afford more leeway for shipping than there is in most of the world's largest ports. The effect of a bridge in reducing the excessive ferry traffic was cited as more than offsetting the interference a bridge might cause; the completion of the three highway bridges now under construction on upper and lower arms of the Bay (Carquinez, Antioch and Dumbarton) will further congest the already inadequate auto ferry facilities. Although figures were cited to show that ferry congestion leads automobile tourists to avoid San Francisco, last year's auto ferry traffic amounted to more than 1¼ million vehicles. In that same period the passenger ferries carried 44,200,000 passengers.

Speaking of the Telgraph Hill site for which the Navy had shown a preference, J. V. Davies, consulting engineer, New York City, said the foundations would be at a depth exceeding the limit for air pressure, that a bridgehead on top of Telegraph Hill was unsuitable and that the length of the approaches required would be prohibitive. He said further that to withhold approval of some feasible crossing on San Francisco Bay would be "a stifling of the port of San Francisco in its foreign and domestic commerce."

Tunnels, on which there has been less restriction as to location, could be built on short, direct routes, but the cost would be higher, about twice the cost of the cheapest bridge project. Proponents of the bridge project in question declared themselves able to finance their $20,000,000 structure and proceed with construction, showing a feasible financial plan with tolls from only the present auto ferry traffic. In contrast to the low cost of this bridge, the Telegraph Hill-Goat Island structure with approaches and terminals, was estimated to cost $72,000,000.

While some of the schemes submitted did not require any modification of the War Department's previous rulings, others did. A large majority of the evidence presented was in agreement on the need for approval of some feasible crossing.

Those who opposed any change in the present rulings included a representative of the commandant of the naval district, representatives of shipping interests and several organizations of shipowners. Of the latter several opposed any structure that would interfere with navigation in any way, but urged that, if a crossing of some sort was deemed a public necessity, preference be given to a long-span high bridge from Telegraph Hill to Goat Island, which would offer the least interference with water traffic.

The data received is being transmitted to the War Department at Washington for final decision.

AT RECENT WAR DEPARTMENT HEARING

Roadway Width Number of Tracks Total Length of Crossing	Estimated Cost (Dollars)	Remarks
40-ft. roadway; to El. Ry. tracks; 1,800 ft. shore to shore	60,000,000 to 70,000,000	15 min. from Market St. to 7th and B'way, Oakland 3½% grades
38-ft. roadway; B. & C. include tracks 27,750 ft.	A—20,000,000 highway B—30,000,000 highway and El. Ry. C—50,000,000 highway, El. Ry. and Steam R.R.	Have plans for combined bridge and tube if high bridge not approved
60-ft. roadway; 4 tracks; Total length, 6 mi.	35,000,000	R.R. figured for Coop 's E-70
54-ft. roadway; 3 rapid transit tracks; Total length, 12 mi.	65,000,000	20 minutes for the trip
Width, 114 ft.; El. and Steam R.R. tracks		
Two 40-ft. roadways Six R.R. tracks	72,000,000, including terminals and approaches	Cost estimate includes elevated tracks on Kearny St. and Grant Ave.
Six traffic lanes Four R.R. tracks	Suspension, 40,000,000 Cantilever, 50 to 75,000,000	Suspension alternative would have three spans, two 2,425 ft. and one 4,850 ft.
Six traffic lanes; 2 El. Ry., 2 Steam R.R. tracks. Length 21,000 ft.		
Two lanes of traffic; two R.R. tracks Tunnel length, 5½ miles	12,000,000	4% grades
60-ft. roadway; 4 tracks		
Length, 6½ miles	40,000,000	
50-ft. roadway; two R.R. tracks (Provision for two more tracks)		
26-ft. roadway Two or more 18-ft. R.R. tubes Total length 25,000 ft.	25,000,000 to 35,000,000	0.4% grade on West end 0.9% grade on East end

Building Congress Urges More Winter Construction

The New York Building Congress has sent out a circular prepared by its committee on seasonal operations, in which it draws attention to the results of its investigations and those of the Department of Commerce which show that, labor productivity is greater on winter building and labor costs are less per unit of building. The congress urges a more general resort to winter construction and suggests that plans to this end be made now. The seasonal operations committee has found that:

(1). The direct additional costs due to construction carried on in cold weather are but a small percentage of the total cost of a building.

(2.) Such direct winter costs are more than offset by savings in other ways.

(3.) These conditions extend down the scale at least to operations as low as $100,000.

The indirect savings which more than counterbalance winter costs may be enumerated as follows:

(1.) Labor bonuses are eliminated.

(2.) Labor turnover is reduced.

(3.) Spread in overhead expense of contractors throughout the year reduces organization and equipment costs.

(4.) Tendency of contractors to lower their margin of profit with the idea in mind of securing sufficient work to keep their organization intact during the winter months.

(5.) Seasonal discounts on materials.

(6.) Seasonal rates by transportation companies in order to relieve the congested periods.

(7.) Saving of interest and taxes on investments lying idle.

(8.) Earlier returns on investment.

The committee reports add that, "It is true, of course, that if other conditions remain the same, winter weather adds the costs of shelter, heating, removal of snow and ice and the like and extreme cold weather lessens efficiency in certain types of manual operations. But in this connection it should be noted that official weather reports show that the number of days during which such unfavorable conditions exist are not as numerous as usually thought to be by the average person."

Denver Plans Six Million Dollar Improvement Program

The budget for 1926 in Denver, Col., calls for six million dollars. Three million of this amount is for the Park Hill sewerage system and $1,500,000 for the paving of 35 miles of streets. The rest for alley paving. During 1925 there has been 33 miles of street paving and 23 miles in alleys.

Engineer Appointed to Committee on Wood Utilization

Dudley F. Holtman, construction engineer, National Lumber Manufacturers Association, Washington, D. C., has been appointed assistant director of the National Committee on Wood Utilization. Mr. Holtman will take up his new duties on Jan. 1. In co-operation with the Forest Products Laboratory, the Bureau of Standards, the Underwriters Laboratory, and the Structural Service Bureau, the National Lumber Manufacturers Association has carried on research and tests, under Mr. Holtman's direction, to better understand the physical and mechanical properties of different species of wood. Mr. Holtman's new work will be along the same lines.

Progress on Additional Water Surveys for Rochester, N. Y.

Continuation of the surveys now being made for an additional water supply for Rochester, N. Y., the study to be limited hereafter to Honeoye and Conesus Lakes, was agreed upon at a conference of officials of the city recently called by Mayor C. D. Van Zandt. The engineers in attendance at the conference were Harold W. Baker, commissioner of public works, C. Arthur Poole, city engineer, Beekman C. Little, superintendent of water-works, and John F. Skinner, deputy city engineer. Purification of the additional supply rather than an attempt to preserve its purity by acquisition of land bordering on the lake shores is reported as likely. New conduits will be required.

Railway and Town Construction Involved in Mine Opening

Close to $20,000,000 is to be spent in developing the Flin Flon copper mine in Northern Manitoba, recently purchased by H. P. Whitney and associates, of New York, according to reports from Ottawa. The Flin Flon mine is located 118 miles from The Pas, Manitoba, to the northwest on the boundary line of Manitoba and Saskatchewan. Among other things, the taking over of the mine will involve the construction of a railway at an estimated cost of more than $2,000,000, and the building of a town to accommodate about ten thousand people. Between $4,000,000 and $5,000,000 is to be spent in developing power on the Churchill River, about 70 miles west of the property.

Building Collapse Shows Lax Inspection

San Francisco Building Collapse Kills Four—Only Seven Inspectors in City

Action on the inadequacy of San Francisco's building department and particularly on lax building inspection was precipitated by the collapse of a brick wall on Dec. 15 which resulted in the death of four workmen. The grand jury found the collapse due to "insufficient inspection by the department of public works." As a result of this incident public sentiment was aroused and the Board of Public Works requested the Board of Supervisors for appropriations to provide for additional engineers and inspectors in the building department.

There has been no secret about the unsatisfactory building situation that has existed in San Francisco for some time. A terse summary of conditions was made in an article by E. T. Thurston, a consulting structural engineer of San Francisco, published in "The Magazine of Western Finance," in which he said "Any conscientious building contractor of experience will testify that buildings exist and are being built all over the city (San Francisco) that constitute a positive public menace in an earthquake zone; and structural engineers will bear evidence to the inadequacy of the building law and its enforcement." . . . "In this city the administration of the building law is well nigh a joke and a byword. Think of it! With three or four hundred plans for buildings filed annually the department of building inspection employs two non-technical office men and two clerks, with the occasional assistance of a consulting engineer paid on a part-time basis. Incidentally this engineer maintains a private office and depends for his living mainly on private practice. Moreover, with an average of over a thousand major buildings (exclusive of residences) under construction all the time, the department has but seven outside men charged with seeing that the buildings are constructed according to law." : . . "Two-thirds of the money collected in permit fees is not expended to support the department of building inspection, but is diverted into other channels."

On Aug. 18 the San Francisco section Am. Soc. C. E., recognizing the danger in the building construction trend and after discussing the duty of an engineering society to inform the public of an epidemic of poor construction, voted $500 for the use of a committee of structural engineers to be appointed by the president, to make a comprehensive investigation of the building situation, giving particular attention to materials and workmanship, the inspection of buildings and the enforcement of the building ordinance. The committee appointed consists of John B. Leonard, chairman, H. J. Brunnier, M. C. Conchot, H. D. Dewell and L. H. Nishkian. This committee has been working with committees appointed for a similar purpose by the San Francisco Chapter of the American Institute of Architects, the San Francisco Industrial Association, San Francisco Builders' Exchange, San Francisco Chapter American Association of Engineers and the San Francisco Building Contractors' Association.

Joint meetings have been held for the most part once a week over a three-month period and after careful and thorough study of the whole situation a letter was sent to the board of public works under date of Dec. 21, signed by representatives of all these organizations and embodying the results of the committee's work in the form of definite recommendations. The letter urged the reorganization of the building department, particularly the employment of additional engineers and inspectors and the appointment of a commission, without pay, consisting of an architect, an engineer and a contractor to serve the board of public works in an advisory capacity and to hear appeals from decisions of, or complaints against, that department. Action on these suggestions had not yet been taken when this article was written.

American Equipment for African Harbor Work

Plant, tools, equipment and all necessary supplies to construct a harbor, breakwater, wharves, roads, etc., for the work which the Firestone Plantation Co. is doing for the government of Liberia at Monrovia started from this country on Dec. 19 on a ship sailing direct to Liberia. On another ship sailing by way of England is going a construction organization sent out by the J. G. White Engineering Corporation which has the contract for the work. Construction work will be started immediately upon arrival. Native labor will be used for practically all the work.

Washington Notes

(*Concluded from p.* 1086)

this case, would deprive the applicant of his legal rights. I do not consider that I have such authority. Such a refusal, in effect, would nullify the provisions of the federal Waterpower Act. So long as the Waterpower Act remains in force, as now worded, I must continue as I have in the past."

THE Joint Board which approved the numbered highway system has been disbanded, but the executive board of the American Association of State Highway Officials has been designated as the continuing body to succeed it. While this board has no authority to add to the system it can arrange the minor corrections and look after the proper mapping and marking of the routes. The map of the numbered system will be ready for distribution early in February.

The Board is doing all it can to encourage the counties and townships to adopt the same form of directional and danger signs as are to be used on the numbered federal aid routes. Several states already have let contracts for signs for county roads.

Engineering Societies

Calendar

Annual Meetings

AMERICAN ROAD BUILDERS ASSOCIATION, New York City; Annual Convention, Chicago, Ill., Jan. 11-15, 1926.

AMERICAN ENGINEERING COUNCIL, Washington, D. C.; Annual Meeting, Washington, D. C., Jan. 14-15, 1926.

ASSOCIATED GENERAL CONTRACTORS OF AMERICA, Washington, D. C.; Annual Meeting, Portland, Ore., Jan. 19-23, 1926.

AMERICAN SOCIETY OF CIVIL ENGINEERS, New York City; Annual Meeting, New York City, Jan. 20-22, 1926.

ENGINEERING INSTITUTE OF CANADA, Montreal: Annual Meeting, Toronto, Ont., Jan. 27-29, 1926.

AMERICAN CONCRETE INSTITUTE, Detroit, Mich.; Annual Meeting, Chicago, Ill., Feb. 23-26, 1926.

The New York Section of the American Water Works Association will hold a meeting January 21, 1925 at the Hotel Pennsylvania.

The Dayton Section, of the American Society of Civil Engineers held its annual meeting Dec. 14; officers for the coming year are: J. K. Grannis, president; F. J. Cellarius and N. Q. Sloan, vice-presidents; and C. H. Eiffert, secretary-treasurer.

The Minnesota Section, American Water Works Association, at its fifteenth annual meeting at the University of Minnesota on Nov. 13, elected the following officers: Chairman, J. A. Childs, engineer, division of sanitation, state board of health; vice-chairman, Felix Seligman, chief engineer of pumping stations, water and light department, Duluth, Minn.; secretary-treasurer, Arthur F. Mellon, filtration engineer, water works department, Minneapolis, Minn.

Personal Notes

R. E. KEOUGH, assistant engineer maintenance-of-way on Eastern Lines of the Canadian Pacific Ry., has resigned to take charge of the Chicago office of the American Fork & Hoe Co., Railway Appliance Division. Mr. Keough has been assistant engineer maintenance-of-way on the Canadian Pacific since 1915. Previous to that time he was with the Chicago, Burlington & Quincy R.R. at Aurora, Ill., first as roadmaster and then as assistant trainmaster.

JOHN A. GILES, at present mayor of Binghamton, N. Y., and for some years its city engineer, will close his term as mayor on Jan. 1 and will resume active duties with the firm of Hoadley & Giles with which he was formerly connected. Mr. Giles was appointed city engineer of Binghamton fourteen years ago and served eight years; he then was made commissioner of public works, and served four years; two years ago he was elected mayor.

J. P. BROOME, civil engineer and present city manager of Salem, Va., has been appointed by the city council of Summit, N. J., as city supervisor of Summit, selection being made from among forty-two applicants from all over the country. Mr. Broome is a native of Maryland and was educated in civil engineering at the University of Maryland. His first service was with the Baltimore & Ohio R.R., especially on location. Then a period with the Ryan & Reilly Co. on Passaic Valley sewer construction was followed by a year of construction on the Delaware, Lackawanna & Western R.R. through South Orange, N. J., for a railroad contractor of New York City. For the next six years Mr. Broome was associated with DuPont de Nemours & Co., of Wilmington, Del., on construction and experimental work. Later he was selected for the city managership at Salem, Va.

CHARLES KINNEY, who a year ago retired as Elkhart County Surveyor and whose term as city engineer at Goshen, Ind., will expire Jan. 1, has resigned as engineer on five county highway units under construction in Elkhart County and on Jan. 1 will become city consulting engineer at Mishawaka, Ind.

WILLIAM GOLDSMITH, consulting engineer for the Riverdale Construction Co., New York City, has been appointed commissioner of public works of Yonkers, N. Y., under the new city administration. Mr. Goldsmith previously had been assistant engineer, superintendent of the Manhattan Asphalt Plant and district engineer for the United States Government in the Philippine Islands.

Obituary

FRANK A. MERRILL, chief engineer of the Boston & Maine R.R. since June, 1924, died at Lynn, Mass., Dec. 21 after a long illness. Mr. Merrill was born in Concord, N. H., Sept. 1, 1857. He received his technical education at Dartmouth College and in 1874 became associated with Charles O. Foss and C. G. Lund on engineering work in New England. He received his early railroad experience on the Concord R.R., the Boston, Concord & Montreal and the Northern R.R. After the death of Mr. Lund, Mr. Merrill and Mr. Foss continued the practice of the firm, largely in railroad engineering. Mr. Merrill was chief engineer of the Concord R.R. from 1885 to 1890 and of the Concord & Montreal R.R. until it was merged with the Boston & Maine R.R. in 1895, when he became assistant chief engineer of the latter road. From 1911 to 1914 he was division engineer in charge of the southern division and in 1914 was appointed engineer maintenance-of-way, in which capacity he served until his appointment as engineer in 1924.

GEORGE THOMAS PRINCE, a retired civil engineer, died at Denver, Colo., on Dec. 18. Mr. Prince was born in Boston, Mass., in 1854, and graduated from Bowdoin College in 1879. Since that time the greater part of his life has been devoted to water supply and sewage works, beginning on the Boston water-works and, after a few years on railroad work, again at Buffalo on sewer construction, and then at St. Paul, Minn., but his longest service was at Atlantic City, N. J., as superintendent of the water-works.

FRANKLIN I. FULLER, first vice-president of the Portland Electric Power Co., died at Portland, Ore., Dec. 16. Mr. Fuller was born in Providence, R. I., May 29, 1858, and after graduation from the high school studied engineering while working the city engineer's office at Providence. In 1881 he went to the Wisconsin Central R.R. on location and in 1882 became construction engineer with the New York, West Shore & Buffalo Ry. In 1883 he went to Portland and since that time had devoted himself largely to electric railway construction and development. Since 1906 he had been a vice-president of the Portland Railway, Light & Power Co., in charge part of the time of operations and all the time of all construction, maintenance-of-way and railway structures.

FRANK C. WILLIAMS, engineer and contractor, Easton, Pa., died suddenly of appendecitis Nov. 28, aged 50 years. Mr. Williams, a native of Boston, left that city in 1909 to take charge of extensive sewer work, as engineer in charge of construction, in Louisville, Ky., where he remained six years. Later he was associated with the James Ferry Co., Inc., Atlantic City, N. J. For the past five years, he was vice-president of the Crichfield-Williams Construction Co. at Jersey City. At the time of his death his firm was engaged in the construction of two sewer contracts for the city of Easton.

WALTER E. HASSAM, of Worcester, Mass., known as the deviser of a method of placing paving, died in Worcester, Dec. 26, aged 60 years. Mr. Hassam was born in Northfield, Vt., and graduated from Norwich Academy. He was assistant city engineer of Worcester from 1889 to 1904 and later served as street commissioner. In 1906 he organized a paving company, which later developed subsidiary companies. The special method of paving consisted in laying stone blocks and grouting the blocks with a sand-cement mixture.

ERNEST LUND, of the engineering and contracting firm of Windsor & Lund, Denver, Colo., died at Denver on Dec. 16.

From the Manufacturer's Point of View

A Point of Contact Between Maker and User of Construction Equipment and Materials

Large Valves on the Hume Dam in New South Wales

The free discharge regulators for the Hume Dam on the Murray River in Australia are claimed to be the largest of their kind in the world and consist of seven Larner-Johnson needle type valves with hand and electrical control. Four of the valves have an inlet diameter of 9 ft. and three an inlet diameter of 13 ft., all the seven valves having an outlet diameter of 7½ ft. The maximum head on the valves is 100 ft. The contract for furnishing these valves has been awarded to J. Blakeborough & Sons, Ltd., Brighouse, Yorkshire.

Business Notes

J. G. WHITE ENGINEERING CORP., New York City, has been engaged, by the municipality of Guatemala City, capital of Guatemala, Central America, for the examination and study of certain municipal requirements, including the engineering for a supply and distribution of drinking water, a sewerage system and paving. Engineers assigned to this work have left New York recently for Guatemala.

GENERAL ELECTRIC CO., Schenectady, N. Y., has announced through its president, Gerard Swope, that it has definitely decided to purchase a site for a manufacturing establishment in the city of St. Louis, Mo. The tracts of real estate selected contain in the aggregate about 155 acres, of which all but 11 acres are within the city limits. The property in general lies between the Belt Line of the Terminal Railroad Association and Goodfellow Ave. It also has a frontage on Bircher Ave. "The exact date on which construction will commence has not been determined, and it will depend, of course, upon the future growth and development of the business of the General Electric Co.," Mr. Swope said. "It has not yet been decided what character of apparatus will be manufactured in St. Louis."

BABCOCK & WILCOX CO., New York City, announces that it has purchased the Fuller-Lehigh Co. and its subsidiaries, manufacturers of pulverized fuel and cement mill equipment. Also that it has purchased the Bailey Meter Co., manufacturers of meters and recorders. Both the acquired companies will be operated with the same organization as heretofore with the exception that Col. J. W. Fuller will become chairman of the board of the Fuller-Lehigh Co., E. G. Bailey will become president, and W. E. Douglass, of Fuller-Lehigh Co., will become vice-president in charge of sales.

LAPEER TRAILER CORP., manufacturers of automatic semi-trailers, announces the appointment of J. B. Beattie as Eastern sales manager, with headquarters in New York City. Mr. Beattie was formerly connected with the Autocar Co.

WESTERN MATERIAL CO., Sioux Falls, S. D., has been appointed distributor for the Caterpillar Tractor Co., the Stockland Road Machinery Co., and the D-A Lubricant Co. for the state of South Dakota.

STOCKLAND ROAD MACHINERY CO., Minneapolis, Minn., has appointed the Beckwith Machinery Co. of Philadelphia, to represent it in the eastern and southern sections of Pennsylvania. J. W. Miller has been appointed district sales manager in Iowa. McDonald & Burgman are new distributors in Miami, Fla. A new foreign distributor is the Gippsland and Northern Co-operative Co., Ltd., Melbourne, Australia. Havana Fruit Co., Havana, Cuba, has been assigned the Cuban territory for Stockland machines.

MUNDY SALES CORP., New York City, has appointed E. L. Sparks distributor for the products of the J. S. Mundy Hoisting Engine Co. for the state of Florida, with offices at Miami.

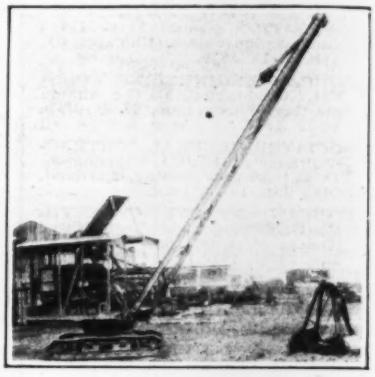
PITTSBURGH-DES MOINES STEEL CO., Pittsburgh, Pa., has opened a new office in Atlanta, Ga., to handle sales in the southeastern states. C. R. Ford, formerly of the Richmond office, is in charge of the Atlanta office.

Equipment and Materials

Half-Yard Crawler for Crane Dragline and Shovel Work

A new ½-yd. crawler machine manufactured by the Mead Morrison Manufacturing Co., Boston, Mass., has been designed with light weight and speed. It is full revolving with a live boom for grab or dragline bucket work and requires only a single re-reeving to change from one type of work to the other.

The boom has a maximum radius of 30 ft. The hoist has three bronze-bushed friction drums 10 in. in diameter provided with single cone asbestos lined frictions and automatic safety brakes. The drum gearing is driven by silent chain from the power plant mounted at the rear of the drums. This power plant is either a Hercules four-cylinder 30-40 hp. gasoline engine or a 30-hp. electric motor. The hoist speed is 150 ft. per min., and the machine develops a swinging speed of 4 r.p.m. Its maximum travel is at the rate of 1 m.p.h. The travel base is carried on two crawler tracks 98 in. over-all width. The treads are all cast

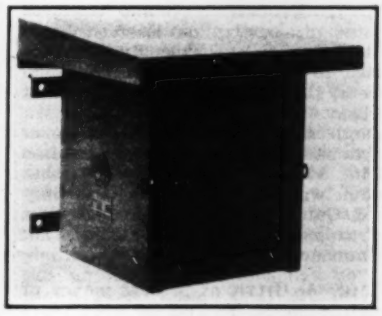


steel of a heavy pattern and are 12 in. wide.

The operating gear consists of five hand levers. Three of these levers control the three friction drums, the fourth controls the travel of the crane, and the fifth controls the shifting from rotating to traveling as well as applying the rotation brake. Two foot levers are provided for the reversing clutches. A single lever controls the cable crowd on the shovel and every operating lever and pedal is so placed that the operator has complete control of his machine without shifting his position.

Magnetic Switch Inclosed in Weatherproof Case

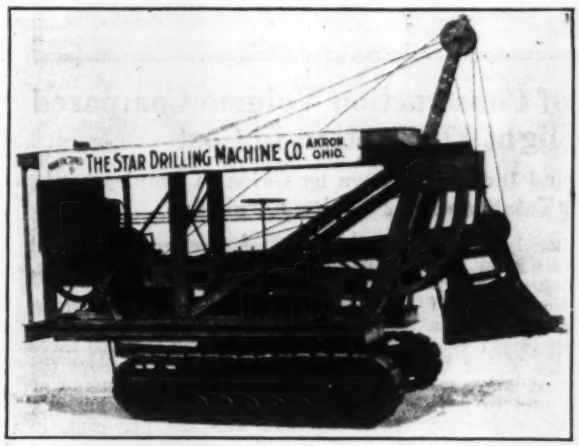
For use in valve control service where operating conditions require that the switch be located in an exposed place or in other similar conditions where it is necessary or desirable to locate the control near the motors, the General Electric Co. has provided a special form of magnetic switch that is inclosed in a weatherproof case and roof. All seams and supports of the outside case are turned to eliminate surfaces where water might collect. The roof is slanting and overhangs the case on all sides to allow water to fall



away. The case itself is provided with special finish to meet the service conditions. A new type of latch is claimed to eliminate possible jamming of the jack strip or any failure to latch tightly due to inaccuracy in the location of the slot.

Skimmer Scoop Full-Revolving Machine of New Type

The new power shovel manufactured by the Star Drilling Machine Co., Akron, Ohio, departs considerably from general conventional design. The scoop handle, which is about 13 ft. long, telescopes into the frame of the revolving deck. This is claimed to be a step forward in power shovel construction, in that it permits a large capacity scoop with a light-weight machine because the weight and load due to shoveling are all concentrated near the center of the machine, thus reducing the necessity for counterweight. The machine, known as their Model 20, is of light weight, full revolving, mounted on full length crawlers and powered by either standard gasoline or electric motors. The weight per square inch of tractor bearing surface is approximately 9 lb., the



total weight of the machine being about 14 tons. The engine and all other controls are located conveniently near the operator.

The shovel is equipped with three types of scoops or buckets—skimmer scoop, pull-in ditcher, and clamshell grab bucket. The skimmer scoop has a capacity of ¾-cu.yd., a crowding motion of about 12 ft., and a digging radius of about 21 ft. The clearance under the bottom of the bucket when dumping at its maximum height is about 17 ft. The mouth of the bucket is 36 in. wide by 36 in. high. The overall length of the machine when the dipper handle is retracted is 20 ft. 2 in., which makes a compact machine where space is at a premium and also a convenient machine when moving through heavy traffic.

By means of a patented device, the scoop can be held in a horizontal position during its entire length of stroke irrespective of the angle of inclination of the boom. Excavations can be effected to a depth of about 20 ft. below the base of the machine. A shallow ditch can be trimmed up with a flat bottom about 14 ft. long for each move of the machine. The scoop teeth always cut a clearance for the rear end of the scoop bottom.

When using the pull-in or ditcher scoop, the rear end of the boom has its pivotal axle clamped fast to the telescoping rack and is arranged so that it can be fastened at a point on the rack selected by the operator as being best suited to his work. When using this ditcher attachment, the machine keeps in advance of the work.

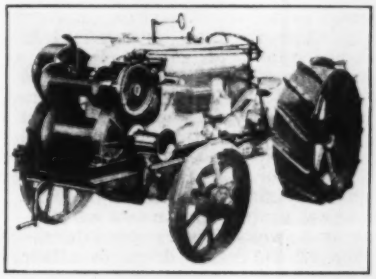
Fordson Accessory Adapts Machine for Use in Deep Snow

The Snow-Motor, consisting of a case of transmission gears attached to the rear of a Fordson tractor differential and a pair of revolving runners, is one of the products distributed by J. S. Innes, 921 Drummond Building, Montreal, Canada. The Snow-Motor is the adaptation of a method developed by a man in Alaska who was freighting with horses and dogs. It was developed first for heavy duty purposes in the mining districts in the northwestern part of the U. S. This was before the war. After the war, it was developed for snow transportation of passengers, freight and for contractors' haulage work in deep snow. The snow-motor is so constructed that it travels *over* the snow rather than through it. The machine complete weighs 5,340 lb. It is claimed that when loaded with five passengers in its cab, it will travel over a 10 ft. snow drift sinking in the lightest snow only about 8 in. Its maximum speed is 12 m.p.h. The machine in addition to carrying 5 passengers in its cab, it is said, can be safely relied upon to exert an average hauling capacity of 5 tons.

The stage coach shown with the snow-motor-equipped Fordson in the illustration is running between Eugene and Bend, Oregon, via McKenzie Pass. It saves a 100-mile detour since the distance through the pass is only 16 miles. The cab carries 14 passengers.

Fordson Hoist Attachment Places Weight on Front Axle

The new hoist, in either single or double-drum type, manufactured by the Pioneer Tractors, Inc., Winona, Minn., presents as one of its distinct features the fact that the operator faces the work at all times. This is made possible by mounting the hoist on the front end of the tractor. The mounting consists of two steel channels secured to the tractor by special bracket castings attaching at the joint between the motor and transmission housing and supported in front by a pedestal cast-



ing through which the front axle kingbolt passes. This mounting places practically the entire weight of the hoist on the front axle and wheels, eliminating strain on any of the tractor castings. The clearance under the tractor is not reduced by this attachment and an extension starting crank is furnished permitting cranking of the motor in front of the hoist. The hoist is quickly and easily attached since there are no holes to drill and no machine work required.

A main housing casting carries the transmission gear case and the bearing housing for the shafting, insuring perfect alignment of all working parts as well as ease of attachment. The main drum is 8 in. in diameter and 14 in. long between flanges rotating on a drum shaft riding on bronze bushings. The clutch is of the double cone type, and all clutch thrust is taken on heavy duty ball bearings and transmitted to

collars resting in grooves in the drum shaft so that no thrust load is transmitted to the housing casting. All bearings, except the ball bearings in transmission case, which are lubricated by splash, are provided with high pressure Alamite fittings.

The standard drive is by a belt 6 in. wide. Slotted holes in the mounting ledges of the housing permit the hoist to slide on the frame, thus permitting adjustment of belt tension. The brake is claimed to be of ample size to hold any load within the capacity of the hoist and is controlled by a foot pedal. One of the special features of the hoist is the placing of lugs or teeth on the inside of the drum flange, thus eliminating the possibility of excessive wearing or cutting of cable since contact with the teeth is impossible. These lugs are engaged by a dog actuated by a control rod from the operator's seat, which provides permanent holding of the load in any position with absolute safety when the crew is required to work under the load.

The main cable speed with standard drive is 140 ft. per minute with a guaranteed lifting capacity of 3,500 lb. on single line for continuous operation. All control levers are within easy reach of the operator. The single-drum hoist complete weighs approximately 800 lb.

Optional equipment consists of a reverse unit which makes possible the driving of the main drum in either direction through a gear shift actuated by control rod from the operator's seat. An auxiliary or second drum assembly is also available which is attached on the front side of the main housing casting —above and slightly back of the main drum.

Publications from the Construction Industry

Diesel Engines—FAIRBANKS, MORSE & Co., Chicago, has made available a new bulletin entitled "Comparison of Diesel Engine Principles" which should be of considerable educational value since it analyzes the various factors which influence the design of Diesel engines. These factors are then interpreted in terms of what they mean to the Diesel engine user in operating economy and dependability. This bulletin is somewhat of a departure from the usual type of sales literature in that it treats a highly technical subject in a way which can be understood by one who has not had an opportunity to make a detailed study of Diesel engines.

Roller Bearings—HYATT ROLLER BEARING CO., Harrison, N. J., has recently issued a bulletin devoted exclusively to roller bearings for industrial equipment, which is known as "Dimension and Load Bulletin No. 1559." After describing the type of Hyatt roller bearings available, formulas are given for determining the size of bearings required as governed by speed, shaft hardness, kind of shaft whether stationary or rotating, and the nature of surface. Tables of basic capacities and dimensions are given for the four types of Hyatt roller bearings.

Business Side of Construction

FACTS AND EVENTS THAT AFFECT COST AND VOLUME

This Week's Contracts—Week Ago—Same Week Last Year

THE money value of contracts reported in the present issue of *Engineering News-Record* is here compared with the figures for corresponding weeks. Minimum costs observed are: $15,000 for water-works and excavations; $25,000 for other public works; $40,000 for industrial and $150,000 for commercial buildings.

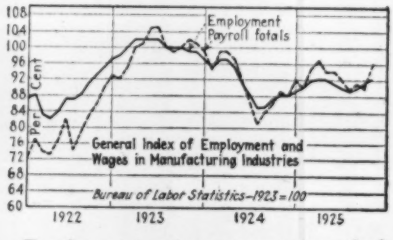
Money Value of Contracts Let—Entire U. S.

Week Ending	Public Work	Private Work	Total Contracts
December 31, 1925	$8,995,000	$10,545,000	$19,540,000
December 24, 1925	16,412,000	58,307,000	74,719,000
December 25, 1924	14,120,000	32,806,000	46,926,000
Heaviest Week			
1925, Sept. 3	16,215,000	69,424,000	85,639,000
1924, May 8	26,370,000	38,829,000	65,199,000
Total for the Year			
1925	1,037,252,000	1,522,191,000	2,559,443,000
1924	953,919,000	1,045,371,000	1,999,290,000

Upward Trend of Construction Volume Compared With Slight Fluctuation in Cost

Expansion in Trade and Industry Shown by Gains in Employment, Pay-roll Totals and Per Capita Earnings

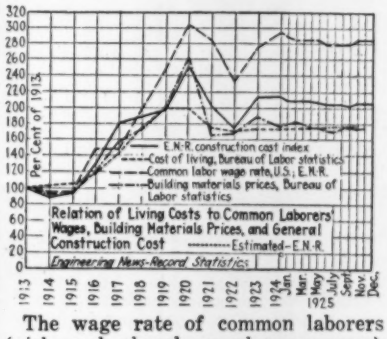
EMPLOYMENT of labor is over 8 per cent greater than a year ago throughout the entire country, according to latest reports of the Bureau of Labor Statistics. Factory pay-rolls show a gain of more than 14 per cent and per capita earnings are higher by 5½ per cent, than at this time last year. The comparatively large gain in pay-roll totals indicates a higher average hourly wage rate in the mills, shops and factories of the nation, particularly for skilled workers. Hours of work per week have an obvious tendency to shorten in inverse proportion to the gain in volume of employment; this principle is in operation at the present time, especially in the trades where there is scarcity of trained mechanics.



Employment gains were most marked in the Eastern and Southern states, while decreases were reported in sections of the West.

In view of these marked indications of expansion in trade and industry, it is estimated that the cost of living is a scant 2 per cent above this time last year. Living costs referred to, would comprise such items as food, clothing, shelter, fuel, light, etc.

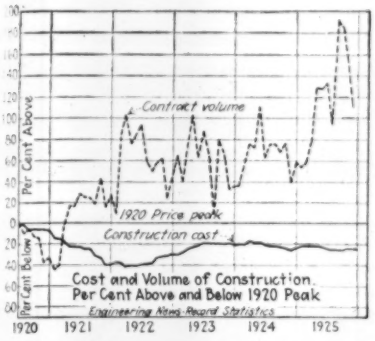
Wages of building trades mechanics, which have been consistently above the general wage rate during the past three years, are now 133 per cent above the 1913 level, according to the Bureau of Labor Statistics. In 1924 the building trades average was 124 per cent, in 1923, 107 per cent, and in 1922, 87 per cent above the 1913 level.



The wage rate of common laborers (pick and shovel or clean-up men) appears in the accompanying illustration, to be considerably above the cost of living. This is due to the fact that in 1913, the base year, the common laborers' rate for the nation was small, compared with living costs at that time.

Cost of construction during the past two-and-a-half years, has hovered about a mark, 20 per cent below the 1920 peak of inflation, occurring in June of

that year. The trend of construction volume during that period was abruptly upward.



Two Large Foreign Jobs Awarded to American Firms

Construction work on the harbor improvement contract awarded to the J. G. White Engineering Corporation by the Firestone Plantations Co. for the Government of Liberia, at Monrovia, West Africa, will begin immediately upon arrival of the equipment shipped Dec. 19, 1925. The project involves a harbor, breakwater, wharves, roads, etc., using practically all native labor.

The Western Union Telegraph Co. has awarded contract to Stone & Webster, Inc., of Boston, for design and construction of a housing development for its cable station forces at Fayal Is. in the Azores.

Pipe Demand During 1925 In Various Sizes

In 1925 Construction News reported work calling for 805,086 lin.ft. of riveted or lock bar pipe. In addition, 98,434 ft. of "steel pipe" was mentioned. The total, 903,520 ft. was of the following sizes:

Inches	Lin.Ft.	Inches	Lin.Ft.
4 to 18	264,350	48	86,199
22	35,620	50	8,996
24	223,770	54	2,532
36	166,900	56	7,380
38	7,300	60	93,800
42	5,793	66	880

The pipe was used chiefly in waterworks, with irrigation second, and oil lines third.

Continued Drop in Bond Issues for State and Municipal Improvement

STATE and municipal bond sales continued small during November according to the *Commercial and Financial Chronicle*. The aggregate for this month was $63,250,071 compared with $74,765,203 of November a year ago. The November total fell far below that of October which shows a total of $78,888,047. The amount of bonds placed during November is the smallest for that month since November, 1922.

There were very few bonds for large amounts placed during November. Some of the large issues are: $6,100,000 loan 4¼s, Philadelphia; $2,890,000 water, school and improvement 4¼s, Paterson, N. J.; $2,325,000 4¼s, Erie County, N. Y.; $2 304,000 4¼s, Trenton, N. J.

The rates in the accompanying table ranged from 4 to 6 per cent. Very few were placed at par or below.

Bond issues for state, county, township and municipal improvements showed a steady upward trend for the month of November, from 1917 to 1921. During the latter year, a period of depression, money was naturally diverted from industrial investments into long term securities.

Since 1923, however, this particular month has shown a downward tendency in state and municipal issues.

The present trend may continue, for the reason that public improvements, held up by the war, have practically all been attended to. Future engineering construction will therefore represent actual expansion in demand.

Representative Public Bond Sales, November, 1925

State	Purpose	Amount	Rate per Cent	Sold For	Basis	Dated	Maturity	Purchased by
Delaware	Highway	$200,000	4	96.869	4.165	Jan. 1	1965	Barr Brothers & Co., Inc., of New York
Michigan	Roads	142,000	4¼	.....	....	Nov. 2	.....	Detroit Trust Co. of Detroit
County								
Ashtabula, O.	Road Impvt.	30,700	5	103.43	4.26	Oct. 1	1927-34	Geo. H. Burr & Co. of Chicago
Armstrong, Pa.	Bridge	126,000	4¼	100.00	....	Nov. 2	1926-30	Armstrong County Trust Co.
Clark, Ind.	Road	16,000	5	103.11	4.42	Sept. 8	1927-36	La Plante-Welsh & Risacher
Columbiana, O.	Road	20,000	5	101.22	5.76	Nov. 15	1927-31	A. T. Bell & Co. of Toledo
Cook, Ill.	Hospital	500,000	4	97.93	4.27	June 1	1926-45	First Trust & Savings Bank of Chicago
	Court House	500,000						
Cuyahoga, O.	Road Impvt.	204,826	4¼	.....	4.245	Aug. 1	1926-35	Guardian Trust Co. of Cleveland
Decatur, Ind.	Highway Impvt.	5,200	4¼	.....	....	Oct. 15	1926-35	Thomas D. Sheerin & Co. of Indianapolis
De Kalb, Ind.	Highway Impvt.	32,100	4¼	101.10	4.265	Oct. 15	1926-35	National Bank of Auburn and others
Delaware, N. Y.	Highway	150,000	4.30	100.03	4.29	Nov. 1	1926-55	Sherwood & Merrifield, Inc., of New York
Daviess, Ind.	Road	11,000	5	102.59	4.52	Nov. 15	1927-36	Meyer-Kiser Bank of Indianapolis
Erie, N. Y.	Penitentiary	1,000,000	4¼	100.364	4.21	Oct. 1	1936-54	B. J. Van Ingen & Co. and others
	Home and infirmary	750,000						
	Highway	200,000						
	Park	200,000						
	County Buildings	175,000						
Essex Fells, N. J.	Water	104,000	5	104.47	4.67	Nov. 1	1927-64	M. M. Freeman & Co. of Philadelphia
Fountain, Ind.	Highway Impvt.	12,500	4¼	100.38	4.43	Nov. 15	1927-36	Fletcher American Co. of Indianapolis
Fulton, O.	Road	39,314	5	101.55	4.54	Sept. 1	1927-31	Detroit Trust Co. of Detroit
Garfield, Okla.	Road	200,000	..	111.11	....	.....	5 yrs.	Enid Bank & Trust Co.
Gibson, Tenn.	Road	500,000	5	.....	....	Nov. 1	1936-45	J. W. Hillman & Co. and others
Harrison, Miss.	Highway and bridge	825,000	5¼	100.99	....	.....	.....	Mississippi Mortgage & Bond Co. of Gulfport
Jackson, Tenn.	Road	11,000	5	100.00	....	Oct. 1	1955	Bank of Gainesboro
Jefferson, O.	Road	110,782	5	102.21	4.59	Dec. 1	1927-35	.....
New Castle, Del.	Highway Impvt.	200,000	4¼	100.30	4.46	Dec. 1	1930-39	National City Bank of New York
	Bridge Impvt.	50,000	4¼	100.30	4.48	June 1	1949-59	
Towns' in								
Chester, N. J.	Improvement	40,000	5¼	.....	....	Aug. 1	1931	M. M. Freeman & Co., of Philadelphia
Jackson, N. Y.	Road Impvt.	6,000	5¼	101.74	5.06	July 1	1927-32	Durfee, Niles & Co. of Toledo
Municipal								
Apex, N. C.	Water	33,000	5¼	100.30	5.48	Sept. 1	1928-60	Hanchett Bond Co., Inc., of Chicago
Arkansas City, Kan.	Paving Impvt.	19,065	4¼	.....	....	June 1	1926-35	First Trust Co. of Wichita.
	Sewer Impvt.	11,604						
Belhaven, N. C.	Paving	37,000	6	101.40	.....	.....	.....	Hanchett Bond Co., Inc. of Chicago
Bergenfield, N. J.	Sewer	143,000	5	100.87	4.935	Nov. 1	1927-65	R. M. Grant & Co., Inc. of New York
Bloomfield, N. J.	Water	557,000	4¼	100.54	4.38	Dec. 15	1927-65	Lehman Bros. of N. Y. and others
	Improvement	60,000	4¼	100.78	4.34			
	Library	158,000	4¼	101.80	4.37			
Charleston, S. C.	Paving	228,000	5	102.21	4.61	Dec. 1	1927-36	Barr Bros. & Co. of N. Y.
Coal Grove, O.	Street Impvt.	6,425	6	102.64	....	Sept. 1	1926-35	A. E. Aub & Co. of Cincinnati
Des Moines, Ia.	Park	12,000	5	100.62	4.78	Sept. 1	1927-29	Polk, Corley & Wheelock Co. of Des Moines
East Rockaway, N. Y.	Park Impvt.	1,000	4¼	100.89	4.585	Nov. 1	1926-38	East Rockaway National Bank
	Fire Apparatus	21,000						
	Village Hall Site	13,000						
Huntingdon, Pa.	Highway	20,000	4¼	101.20	4.38	Sept. 1	1930-49	Grange Trust Co.
Lake City, Fla.	Paving	120,000	6	100.79	5.83	Dec. 1	1926-35	Florida National Bank of Jacksonville.
Paterson, N. J.	Water	1,887,000	4¼	101.94	4.355	Dec. 1	1926-65	Bankers Trust Co. of N. Y. and others
	School	866,000		102.6	4.35			
	Improvement	137,000		101.19	4.38			
Waterloo, N. Y.	Paving	16,000	4¼	100.729	4.64	Nov. 1	1926-41	R. F. De Voe & Co. of New York

Weekly Construction Market

VALUE of contracts let throughout the entire country for the year 1925 shows a gain of 28 per cent compared with last year's total.

The 1925 yearly average of construction cost is nearly 4 per cent below that of 1924; present costs are a trifle over one per cent below December, 1924.

Present trend of construction is therefore in strict conformity with the rule that lower cost tends toward greater volume.

Regardless of the present demand for materials in Florida, the only serious danger of increased construction cost lies in the direction of the skilled trades.

It is evident from proposed demands for higher wage schedules in some of the principal centers, that the skilled mechanics anticipate another big year for 1926.

The current building materials market in New York City shows marked firmness over a week ago in brick, lumber, cement, wire nails and lath. This trend prevails throughout the state with the exception of a noticeable weakness in dimension lumber and aggregates in the northern cities.

A comparison of current prices with those in effect at this time last year, shows that in the majority of the nine cities reporting weekly to *Engineering News-Record*, prices are lower for steel products (except wrought pipe), cement, aggregates, c.-i. pipe, fir timbers, lump lime, hollow tile, linseed oil, and common laborers' wages. Pine timbers and brick are higher.

	New York	Atlanta	Dallas	Chicago	Minneapolis	Denver	San Francisco	Seattle	Montreal
Steel Products									
Structural shapes, 100 lb.	$3.34	$3.40	$4.15	$3.10	$3.35	$3.07½	$3.30	3.35	$3.75
Structural rivets, 100 lb.	4.50	3.50	4.75	3.50	3.75	4.65	5.00	3.75	5.50
Reinforcing bars, ¾ in. up, 100 lb.	3.24	3.30	3.38	3.00	3.25	3.77½	3.35	3.25	2.30
Steel pipe, black, 2½ to 6 in. lap, discount	48%	57%	53.6%	51%	54.25%	36%	39.2@51%	45%	59.49
Cast-iron pipe, 6 in. and over, ton	51.60@52.60	46.00	54.00	50.20@51.20	5.00	64.00	—51.00	55.00	55.00
Concreting Material									
Cement without bags, bbl.	2.50@2.70	2.25	2.05	2.10	2.32	2.84	2.31	2.65	1.80
Gravel, ¾ in., cu.yd.	1.75	2.20	2.38	2.00@2.20	1.65	1.90	2.15	1.50	1.25
Sand, cu.yd.	1.00	1.65	2.00	2.25	1.25	1.00	1.50	1.50	1.25
Crushed stone, ¾ in., cu.yd.	1.85	2.20	2.83	2.00@2.20	1.75	2.50	2.15	3.00	1.50
Miscellaneous									
Pine, 3x12 to 12x12, 20 ft. and under, M.ft.	62.00	34.00	56.00	57.00	36.75	49.50	27.00	25.00	60.00
Lime, finishing, hydrated, ton.	18.20	22.50	20.00	20.00	25.50	24.00	22.00	24.00	21.00
Lime, common, lump, per bbl.	2.25@2.75	1.30	1.85	1.30	1.35	2.70	1.60	2.80	10.00
Common brick, delivered, 1,000.	+18.00@20.00	10.50	11.10	12.00	13.50	12.00	15.50	15.00	17.50
Hollow building tile, 4x12x12, per block.	Not used	.0893	.10	.075	.....	.075	.....	.10	.12
Hollow partition tile 4x12x12, per block.	.1112	.0893	.10	.075	.07	.075	.108	.10	.10
Linseed oil, raw, 5 bbl. lots, gal.	—.93¾	1.07	1.13	—1.00	1.08	1.21	1.10	1.12	1.14
Common Labor									
Common labor, union, hour.	.62½@.75	.30	.....	.87½	.....	.50@.55	.55	.62½	.....
Common labor, non-union, hour.	.....	.25	.30@.50	.82½	.45@.55	.35@.50	.50	.50	.20@.30

Explanation of Prices—Prices are to contractors in carload lots unless other quantities are specified. Increases or decreases from previous quotations are indicated by + or — signs. For steel pipe, the prevailing discount from list price is given; 45-5% means a discount of 45 and 5 per cent.

New York quotations delivered, except sand, gravel and crushed stone, alongside dock; common lump lime, in 280-lb. bbl. net and hydrated lime, f.o.b. cars; tile "on trucks"; linseed oil and cast-iron pipe f.o.b. Labor—Concrete laborers' rate, 93¾c.; building laborers, 75c.; excavating laborers, 62½c. per hr.

Chicago quotes hydrated lime in 50-lb. bags; common lump lime per 180-lb. net. Lumber, sand, gravel and stone delivered on job; pine is 4-in. instead of 3-in.

Minneapolis quotes on fir instead of pine. Brick, sand and hollow tile delivered. Cement on cars. Gravel and crushed stone quoted at pit. We quote on brown lime per 180-lb. net; white is $1.70 for Kelly Island and $1.60 for Sheboygan. Common labor not organized.

THIS limited price list is published weekly for the purpose of giving current prices on the principal construction materials, and of noting important price changes on the less important materials. Moreover, only the chief cities are quoted.

Valuable suggestions on costs of work can be had by noting actual biddings as reported in our Construction News section.

The first issue of each month carries complete quotations for all construction materials and for the important cities. The last complete list will be found in the issue of December 3, the next on January 7.

Denver quotes on fir instead of pine. Cement "on tracks"; gravel and sand at pit; stone on cars; lime, brick, hollow tile and lumber on job. Tile price is at warehouse. Linseed oil, delivered in wooden bbl. Common lump lime per 180-lb. net.

Atlanta quotes sand, stone and gravel per ton instead of cu.yd. Common lump lime per 180-lb. net.

Dallas quotes lime per 180-lb. bbl. Steel, cement, cast-iron pipe and crushed stone f.o.b. cars, other materials delivered.

San Francisco quotes on Heath tile, size 5½ x 8 x 11½. Prices are all f.o.b. warehouses except C. I. pipe, which is mill price plus freight to railway depot at any terminal. Common lump lime per 180-lb. net. Lumber prices are to dealers in yards at San Francisco, for No. 1 fir, common.

Seattle quotes on Douglas fir (delivered) instead of pine. Lump finishing lime per 180-lb. net. Brick and hollow building tile delivered. Hydrated lime in paper sacks. Sand and gravel at bunkers.

Montreal quotes on pine lumber. Sand, stone, gravel and lump lime per ton. Stone and tile are delivered; sand, gravel, lime and cement on siding; brick f.o.b. plant; steel and pipe at warehouse. Hollow tile per ft. Cement price is in Canadian funds (the Canadian dollar stands at 99.93). Bag charge is 80c. per bbl. Discount of 10c. per bbl. for payment within 20 days from date of shipment. Steel pipe per 100 ft. net; 3½-in., $59.49.

Business Briefs

Call money quoted at 6 per cent, Dec. 28; year ago, 5½ per cent.

Time loans: sixty-ninety days, 4¾@5 per cent; four-six months, 5 per cent.

Commercial paper: best names, 4½ per cent; other names, 4¾ per cent.

On December 1, 1925

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For Explanation and Details of Index Since 1913 See the First Issue of Every Month